US Army Corps
of Engineers
Waterways Experiment
Station

Final Report
CPAR-GL-98-1
April 1998

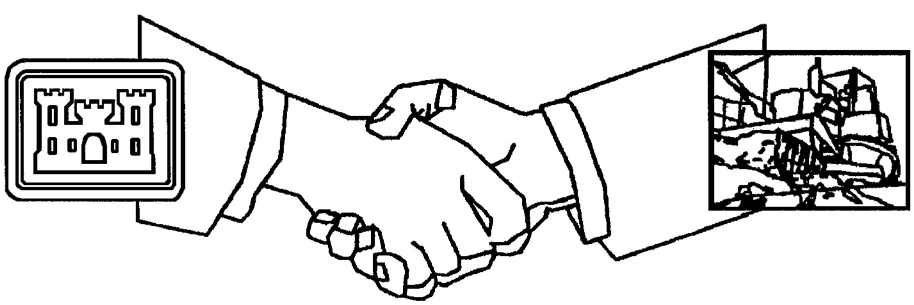
CONSTRUCTION PRODUCTIVITY ADVANCEMENT RESEARCH (CPAR) PROGRAM

Installation of Pipelines Beneath Levees Using Horizontal Directional Drilling

by

Kimberlie Staheli, David Bennett, Hugh W. O'Donnell,
and Timothy J. Hurley

Approved For Public Release; Distribution Is Unlimited



**A Corps/Industry Partnership to Advance
Construction Productivity and Reduce Costs**

Construction Productivity Advancement Research (CPAR) Program

Technical Report
CPAR-GL-98-1
April 1998

Installation of Pipelines Beneath Levees Using Horizontal Directional Drilling

by Kimberlie Staheli, David Bennett, and Timothy J. Hurley

U.S. Army Corps of Engineers
Waterways Experiment Station
3909 Halls Ferry Road
Vicksburg, MS 39180-6199

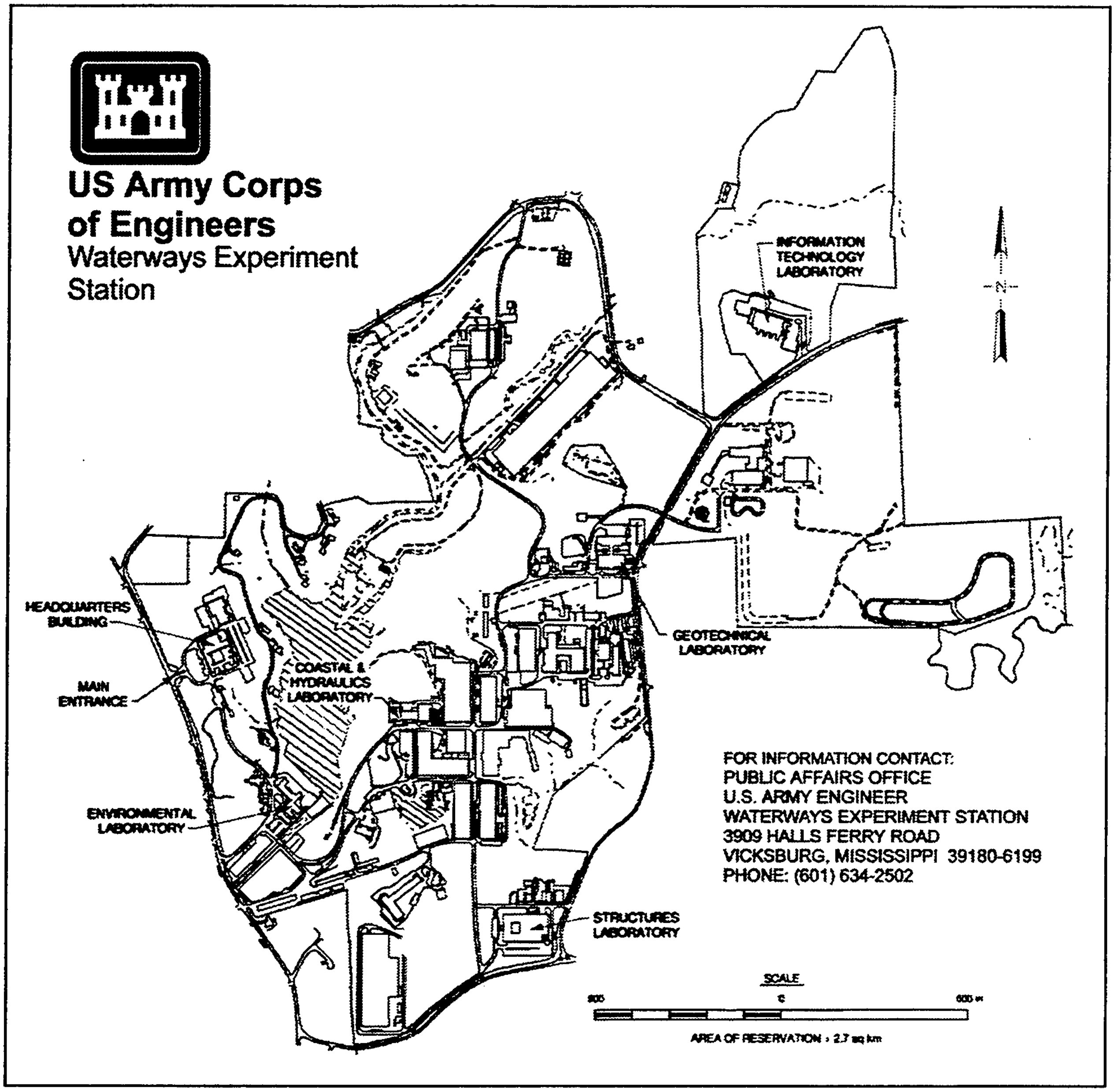
Hugh W. O'Donnell

O'Donnell Associates, Inc.
P.O. Box 1208
Sugarland, TX 77487

Final report

Approved for public release; distribution is unlimited

Prepared for U.S. Army Corps of Engineers
Washington, DC 20314-1000



Waterways Experiment Station Cataloging-in-Publication Data

Installation of pipelines beneath levees using horizontal directional drilling / by Kimberlie Staheli ... [et al.] ; prepared for U.S. Army Corps of Engineers.
194 p. : ill. ; 28 cm. — (Technical report ; CPAR-GL-98-1)
Includes bibliographical references.
1. Pipelines. 2. Levees. 3. Directional drilling. I. Staheli, Kimberlie. II. United States. Army. Corps of Engineers. III. U.S. Army Engineer Waterways Experiment Station. IV. Geotechnical Laboratory (U.S. Army Engineer Waterways Experiment Station) V. Construction Productivity Advancement Research Program (U.S.) VI. Series: Technical report (U.S. Army Engineer Waterways Experiment Station) ; CPAR-GL-98-1.
TA7 W34 no.CPAR-GL-98-1

Contents

List of Tables

Preface

The work reported herein resulted from research and development conducted under the Construction Productivity Advancement Research (CPAR) Program by the U.S. Army Engineer Waterways Experiment Station (WES), Vicksburg, MS, and O'Donnell Associates, Inc., Sugarland, TX. Headquarters, U.S. Army Corps of Engineers, technical monitors for this project were Mr. Donald Pommer (CECW-OM) , Mr. Greg Hughes (CEMP-ET), and Mr. David B. Mathis (CERD-C). Their comments and guidance were very helpful and contributed to the final preparation of this report. This project has demonstrated that Government and industry working together through the Corps' CPAR program benefits the partners and the overall industry.

Cooperation and support were provided by the industry partner, O'Donnell Associates, Inc., and the secondary industry participants, A&L Underground, Inc., P. R. C. International, Louis J. Cappozoli & Associates, Colonial Pipeline Company, Hemphill Construction, Horizontal Drilling International, NASTT, Parchem, Inc., Sunland Construction, Sharewell, Inc., Tulsa Rig Iron, Inc., Trenchless Technology Center, Delft Geotechnics Institute, and the City of Vicksburg. These companies and their staffs are commended for their contributions to this project and the industry.

This research was done under the general supervision of Dr. William F. Marcuson III, Director, Geotechnical Laboratory (GL). The report was written by Ms. Kimberlie Staheli, Research Engineer, Soil and Rock Mechanics Division (SRMD), Mr. David Bennett, Acting Chief, SRMD, Mr. Hugh W. O'Donnell, President, O'Donnell Associates, Inc., and Mr. Timothy Hurley, SRMD.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander was COL Robin R. Cababa, EN. Permission was granted by the Chief of Engineers to publish this information.

Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
degrees (angle)	0.02	radians
feet	0.31	meters
feet per second	30.48	centimeters per second
inches	2.54	centimeters
kips per square foot	4884.31	kilograms per square meter
miles (U.S. statute)	1609.0	meters
pounds	0.45	kilograms
pounds (force) per square foot	4884.4	kilonewtons per square meter
pounds (force) per square foot	478.80	dynes per square centimeter
pounds (mass) per cubic foot	0.16	kilograms per cubic meter
pounds per gallon	0.11	kilograms per liter
pounds per square inch	6.89	kilonewtons per square meter
quarts	0.95	liters
tons	0.91	metric tonnes
tons per square foot	0.01	kilonewtons per square meter
Note: cubic meters	263.95	gallons
feet of pressure head	0.43	pounds per square inch

1 Introduction

Background

Early methods of installing pipelines and utilities across rivers and streams involved excavation of trenches. After the placement of the pipeline, the trenches were backfilled to protect the pipeline from hazards. These early dredged crossings were generally sited at the channel crossing of the thalweg between bends of the river. Here the river is generally a wide, shallow rectangle. This location is chosen due to its hydraulic stability and the economic limitation of the dredging equipment.

In and across the U.S. Army Engineer District (USAED) Lower Mississippi Valley, lies the heart of the pipeline transmission network of the United States. Hundreds of individual pipelines traverse from Texas and out of the Gulf of Mexico across the numerous rivers, bayous, and wetlands of Louisiana to service the northeast population centers on the Atlantic coast. Along the leveed banks of the lower Mississippi River, pipeline crossings exist between nearly every bend-way. The crossing of these earthen flood control structures presents a difficult and expensive construction problem due to concerns about the integrity of the levee against a sliding failure.

Horizontal Directional Drilling Method

In the early 1970's a new process was introduced to install pipelines by use of horizontal directional drilling (HDD) techniques acquired from the oil and gas industry. The method has steadily grown to achieve worldwide acceptance and has been used in over 3,000 installations totaling over (1,288 km) 800 miles of pipelines. Today pipeline installations increasingly rely upon HDD technology as the primary method for crossings of watercourses, wetlands, utility corridors, roads, railroads, shorelines, environmental areas, and urban areas.

The placement of pipelines by the HDD method requires the drilling of a guided pilot bore, generally using a 7.3- to 11.43-cm- (2-7/8- to 4-1/2-in.-) diam drill pipe. At the lead, or downhole, end of the pilot string is a fluid powered cutting tool. The cutting tool is either a drill motor to which a bit is connected or a jet bit with nozzles. Drilling fluid is pumped through the string, and fluid causes the motor to rotate which turns the bit to cut the hole. With jet bits, the velocity

from the jet nozzle erodes the hole in front of the drill pipe. Located behind the drill head is a section of the drill pipe with a small bend or angular deviation. This section, known as a bent sub or bent housing, allows the motor or jet nozzle to be directed. A steering tool is latched onto a locking tool on the drill pipe. In this steering tool are a magnetometer and other devices to determine the azimuth, inclination, and orientation of the tool or tool face. Position determinations are made, and the data from the steering tool is plotted in the field to determine the profile and alignment of the bore. Analysis of this position plot is then used to determine drilling progress and path. At a desired location, the pilot drill pipe exits the ground. The pilot bore is then enlarged by pulling reaming tools back through the bore. Once this operation is completed, the pipeline or conduit is attached to the drill pipe and pulled back through the predrilled bore. This is accomplished as the drill pipe is removed, joint by joint, from the drilled path until the pipeline reaches the ground surface at the entry end of the bore.

One of the primary parameters in horizontal directional drilling is the drilling fluid or mud. The drilling mud is usually comprised of a bentonite and water mixture whose main function is to power the downhole cutting tool used to open the bore. Secondary functions of the drilling mud are to serve as a lubricant for the pipeline during installation and, in cases of rock or hard ground bores, to remove cuttings from the bore.

The use of HDD has been restricted, in part, by major misunderstandings of how the HDD process actually functions. It is assumed by many that it is similar to well drilling or tunneling in that an open bore is required. This is true only in hard geologic materials such as rock. The majority of HDD pipeline crossings installed to date have been performed in soft ground comprised chiefly of alluvial deposits of silts, sand, and clay. In these types of soils the process begins with a small pilot bore from which various cutters are inserted to loosen the soil as it is mixed into a slurry by the injection of the drilling mud. Once this slurry pathway has been made large enough, generally 25.4 to 30.5 cm (10 to 12 in.) greater than the diameter of the pipeline, the installation of the pipeline commences by pulling the pipeline back through the soft slurry pathway. The in situ soil and fluid are then compressed into the formation, and only a small percentage of the soil is actually pumped out of the path.

Problem Identification

Although horizontal directional drilling could offer cost-effective, safe alternatives to installing pipelines with open trenching, the Corps of Engineers (CE) does not have standard guidelines allowing the installation of pipelines with this construction method. As a result, permitting policies are extremely varied and some Districts strictly prohibit the use of this technique. Recommended guidelines for pipeline installation using HDD should be developed for use by the Corps of Engineers' Districts. Without this guidance to provide criteria by which to evaluate proposals for levee crossings, the use of HDD for pipeline installation may not be allowed in areas where the installation technique might be applicable and capable of providing a tremendous cost savings to the pipeline industry and the Corps of Engineers.

Objectives

The objectives of the research are to develop guidelines (Appendix A) for installing pipelines (Appendix B, Delft Geotechnics Report (1997)) beneath rivers and within levee rights-of-way using HDD techniques, without endangering the levees, and to demonstrate that these techniques offer substantial economic and operational advantages over current practices.

Approach and Research Plan

The key elements of this research and development (R&D) were:

a. Determination of the hydraulic forces developed and employed in the HDD process.

b. Determination of the function and effect of drilling fluid during drilling operations.

c. Development of practical procedures to allow the use of HDD method for installing pipelines beneath and adjacent to flood control levees.

One goal of this study was to determine how the drilling fluid or mud used in HDD actually behaved underground, especially beneath an earthen structure such as a flood control levee. To accomplish this task, a two-phase program was conducted. The first phase involved the development of a conceptual model to evaluate machine-ground interaction and stability problems. This approach evaluated various ground conditions, geometries, and machine operational characteristics. The second phase involved full-scale field testing where horizontal drilling operations were employed beneath a levee.

During the course of the drilling, subsurface pressures were monitored by piezometers in vertical boreholes placed at various depths and offsets from the bore center line. All operational pressure levels and mud characteristics were also recorded. During the field tests, three bores were constructed using HDD. After the drilling took place, the directionally drilled path through the subsurface soils was examined by controlled excavation to view any migration of the drilling fluid.

A successful conclusion of the study would be the development of guidelines that would allow the successful and safe use of the HDD method for pipeline and utility crossings of CE flood control levees.

Potential Benefits

The pipeline industry would realize a tremendous benefit from the use of HDD in crossing of flood control levees. This benefit would include significant cost reduction in construction and maintenance presently required for levees and

adjacent road crossings such as bridges, concrete boxes, earthen cover, and ramps. The use of the technique could also benefit the Corps of Engineers by (a) eliminating blockage of levee crown from buried pipelines, pipeline bridges, or conduit boxes, (b) eliminating differential settlement imposed on levees by the construction of buried pipelines, pipeline bridges, or conduit boxes, (c) improving the operation and safety of grass cutting and other maintenance equipment on the levees, and (d) reducing risk of rupture of pipelines located above or near ground surface on levee slopes.

Concurrent Research

The growth of HDD installation for pipelines and other utilities throughout the world has prompted several respected organizations and institutes around the globe to initiate studies into various aspects of HDD for crossings of waterways and other obstacles by pipelines or conduits. Included among these researchers are three industry participants of this Construction Productivity Advancement Research (CPAR) program: the American Gas Association's (AGA) P. R. C. International, the Trenchless Technology Center of Louisiana Tech University, and Delft Geotechnics of The Netherlands. The USACE Waterways Experiment Station (WES) also completed studies on mini-HDD under the CPAR program.

The research performed by these entities is of great value. Of particular importance were publications of the P. R. C. International (formerly the Pipeline Research Committee of the AGA) entitled:

a. "Drilling Fluids in Pipeline Installation by Horizontal Directional Drilling," *Practical Application Manual*, PR-227-9321 (AGA, Pipeline Research Committee 1994).

b. "Installation of Pipelines by Horizontal Directional Drilling," *Engineering Design Guide*, PR-227-9424 (Hair, Cappozoli, and Stress Engineering 1995).

In addition, research completed by the Delft Geotechnics in the 1980's (Luger and Hergarden 1988) was of great significance and is directly related to the research contained in this report. Delft Geotechnics is an internationally acclaimed geotechnical institute and a leader in research into the behavior of underground construction and flow behavior. Shortly after the installation of the first HDD crossing of the Buiten IJ near Amsterdam in early 1984, Delft Geotechnics became involved in the application of research and development of trenchless techniques involving HDD.

A direct result of this research was a large-scale field test program during installation of pipelines by HDD. The results of their initial studies provided a formula for predicting in-hole fluid pressures and presented design recommendations (Luger and Hergarden 1988).

2 Conceptual Model

Development of the Conceptual Model

The goal of this first phase of the research and development was to investigate a conceptual model to evaluate machine-ground interaction and stability problems using numerical modeling methods to evaluate various ground conditions, geometries, and machine operational characteristics.

Review of Hydrofracture Criteria

This limited study evaluated and compared the results of plastic and elastic soil behavior. The study of plastic behavior was based on work performed by Yanagisawa and Komak Panah (1994) who conducted studies evaluating hydrofracture in cohesive soils. Their findings concluded that the pressure required for hydrofracture was a simple function of the cohesion and vertical stress of the soil. The plastic deformation of the soil was solely a function of soil cohesion. The studies on elastic behavior evaluated the ratio between horizontal and vertical soil stresses.

Studies on plastic behavior showed that hydrofracture will occur before plastic yielding at shallow depths. Plastic yielding will occur alone before hydrofracture at great depths. It was also concluded that fluid pressure may cause hydrofracture when the pressure exceeds twice the value of undrained cohesion of the soil, i.e., the unconfined compressive strength of the soil. Therefore, a pressure of 689 kN/m^2 (100 psi) would be expected to cause hydrofracturing in a clay with an unconfined compressive strength less than 689 kN/m^2 (7.2 tsf), a very stiff clay. According to these assumptions, for a compacted saturated clay with a soil unit weight of approximately 19.6 kN/m^3 (125 pcf), 60 kN/m^2 (8.7 psi) for each 3.05 m (10 ft) of depth would be required to cause hydrofracture. However, these studies did not address the propagation of hydrofracture through the soil but focused solely on the pressures required to initiate hydrofracture.

Studies on the elastic behavior compared the stresses on the boundary of the hole and compared these stresses with the tensile strength of the soil. The coefficient of lateral earth pressure was varied to determine its effect on hydrofracture potential. As the ratio of horizontal soil stress to vertical soil stress approached

unity, the stresses required to produce hydrofracture were comparable to those computed for the plastic deformation analysis.

The fact that hydrofracturing has not been observed in many situations where the theoretical criteria have been exceeded makes it clear that important factors have been ignored.

Conclusions of the Conceptual Studies

Because the results of the study were dependent on the hydraulic pressure generated by the drilling apparatus, there were many unanswered questions about the possibility of hydrofracture due to the lack of information on the actual pressures in the bore hole. Use of a finite element model to predict the pressure dissipation and expansion of the borehole was not undertaken due to lack of a set of criteria that adequately dealt with the physics of the drilling environment. Complicating factors with regard to the pressure dissipation around the nozzle of the drilling apparatus would not allow quantification of the pressure in the bore hole prior to field testing. These complicating factors included head loss through the nozzles as well as the energy required to generate the angular momentum of the mud particles as they exit the nozzle and progress down the bore path. To handle such conditions, an advanced fluid mechanics model capable of calculating the effects of turbulent flow would be required. These results would have to be analyzed in combination with the effects of the soils and underground conditions to accurately predict the hydrofracturing potential. Due to the complex nature of the modeling and the vast amount of effort required to couple these analyses, the study was terminated at this point.

3 Details of the Field Experiment

Site Description and Geology

The site was approximately 7.2 km (4.5 miles) south of Mayersville, MS, at Lake Carlisle. Figure 1 is a section of the Whiting Bayou U.S. Geological Survey (USGS) Quadrangle Map showing the project site. The levee was an old landside setback portion of the mainline Mississippi River levee, built in 1941 during a period of rapid channel migration. Upstream channel stabilization measures completed after the setback was constructed were successful and allowed the mainline levee to be reestablished riverward of the setback. Therefore, the landside setback served no flood control function and was being considered for degrading (abandonment). Figure 2 is an aerial view of the test site showing the Mississippi River, the mainline flood control levee, and the setback levee used for the field testing. The site, with plans for abandonment, was ideal for this research because:

a. Any potential damage to the setback levee that might occur during the research would not impair the flood protection capabilities of the mainline levee.

b. The setback levee could be excavated to isolate and analyze post-construction characteristics in the vicinity of the three bores, and reconstruction was not required.

c. The site had been extensively characterized and mapped by the Corps of Engineers. The original geologic maps, aerial photographs, and other data were found in the WES files.

The geology of the site and area has been shaped by the Mississippi River. Lake Carlisle is a remnant of an alluvial channel that was abandoned and partially filled in by sediment. Figure 3 shows a view of Lake Carlisle from the setback levee. Stratification of subsurface soils resulted from the flooding and meandering of the river channel. The current channel of the Mississippi River is approximately 1.6 km (1 mile) west of the project site and is confined by the flood protection levees. The area on the landside of the setback levee is flat with gentle ridges and swales generally less than 0.3 m (1 ft) higher or lower than the surrounding areas. These features resulted from the periodic flooding of the river before levees were constructed.

To complement the existing site geological information, six vertical boreholes were drilled in the immediate vicinity of the planned bores. Figure 4 is a photograph of the boring operations. The plan view of the locations of these borings, later converted to pneumatic piezometers, is shown in Figure 5. The borings were completed to elevations approximately 3 to 7.5 m (10 to 25 ft) below the planned HDD bore paths. The boring logs are shown in Appendix C. Each of the borings was converted to pneumatic piezometers, with the tips of the piezometers at each toe installed at or just above the elevations of the HDD bores. The two borings drilled from the crest of the levee were significantly deeper than the planned directional drilled pipelines; the piezometer tips were 6 to 7.5 m (20 to 25 ft) deeper than the HDD bore elevations. This arrangement was intended to allow evaluation of the effects of drilling fluid pressures above and below the pipeline elevation.

The borings indicated relatively uniform subsurface conditions. The top stratum consisted of 0.6 m (2 ft) of medium to very stiff tan to gray silty clay in all borings except boring 6 where this top stratum was 1.5 m (5 ft) thick. The substratum consisted of fine tan and gray, clayey and silty sands of loose to dense consistency. The consistency generally increased with depth but was predominately medium. Blow counts ranged from 1.5 to 11 blows/m (5 to 38 blows/ft). Occasionally lenses of more silty or clayey sands and sandy clays were encountered. The groundwater levels fluctuated with river levels, but were very near the ground surface.

The postconstruction excavation and autopsy served to confirm the uniform subsurface conditions. As mentioned previously, the excavations also revealed a fairly thin, but relatively continuous, horizontal seam of plastic clay on the landside of the levee approximately 5 to 10 cm (2 to 4 in.) thick and approximately 1.8 to 3 m (6 to 10 ft) below grade. This seam was less evident on the riverside of the levee. The excavations further revealed that the levee was predominantly constructed using the silty sands as borrow material. Very little clay was found in the levee cross section. Soil properties and engineering characteristics are summarized in Appendix C.

Planned Testing Procedures

Three bores were planned beneath the earthfill levee located in Mayersville, MS. The planned length of each bore was approximately 160 m (530 ft). Control points were established prior to construction at the landside and riverside toes of the levee, 7.6 m (25 ft) below the original ground elevation. These control or target points were the planned depths of the pipeline beneath each levee toe. The levee was 5.6 m (18.4 ft) from the natural grade to the crest and 49 m (160 ft) horizontally from toe to toe. For all three bores, a dye tracer was added to the drilling fluid to allow easy identification of any fluid migration paths during the postconstruction autopsy.

The levee was instrumented with pneumatic piezometers at six locations. Two of the six pneumatic piezometers were on the riverside toe of the levee, two were on the levee crest, and two were on the landside toe. Figure 5 shows the location

of each pneumatic piezometer in relation to the levee and the planned horizontal bore alignments. Pressures at each of the piezometers, representative of static groundwater head, were measured prior to the drilling operations to establish a baseline pressure and to determine "normal" pressure fluctuations due to the changes in groundwater levels that occurred with changes in river stages.

During all three pilot bores, drilling mud weight and viscosity were systematically varied. In addition, internal drilling fluid pressures varied throughout each bore to provide a basis for determining the effect of the drilling pressure on the zone of influence around the pipeline. For the drilling of the first bore, mud weights were held relatively constant, and internal drilling fluid pressures, measured at the nozzle in the pipe string, were systematically varied. For the second bore, mud weights were held very close to the mud weights recorded in the first bore, but drilling pressures were substantially different. For the third bore, mud weights were considerably different than on the first two bores, but drilling pressures were held relatively constant over the length of the bore.

Upon completion of the first pilot bore, a 50.8-cm- (20-in.-) diam fly cutter was attached to the drill string, along with a 30.5-cm- (12-in.-) diam steel pipe string. The hole was then reamed and the pipe installed in one pass. Upon completion of the second pilot bore, a barrel reamer was attached to the drill string for reaming; however, no pipe was attached to the reaming assembly. One reaming pass was made on the second bore. Pilot Bore 3 was terminated once the bore had progressed approximately 18.3 m (60 ft) beyond the landside toe of the levee, and no reaming took place.

Once all three bores were completed, an autopsy was performed to expose the drilled bores to determine the zones of influence around each bore. The pink-dyed drilling mud was visually observed during the autopsy, and photographic evidence documented any migration of drilling fluids.

Site Layout/Planned Alignment

Figure 5 shows the planned alignment for the three bores. All three bores originated from the riverside of the levee. The first bore had a planned length of 162 m (530 ft). For the second bore, the drilling rig was relocated at a lateral distance of 7.6 m (25 ft), with Bore 2 parallel to Bore 1. Pilot Bore 3 originated from the same location as Pilot Bore 2; however, the drill rig was shifted to a new angle. Pilot Bore 3 made a turn in the alignment and crossed the levee parallel to Pilot Bores 1 and 2.

Drilling Fluid Properties and Composition

Drilling fluids are very important to the success of directional drilling operations. A properly formulated drilling fluid provides lubricity, effective hole cleaning, and hole stability. The principal functions of drilling fluids as they apply to directional drilling are summarized below:

a. *Transmission of hydraulic power*. The drilling mud transmits power to the downhole mud motor which turns the bit and mechanically drills the hole.

b. *Hydraulic excavation*. Soil is excavated by erosion from the high-velocity drilling fluid that streams from the jet nozzles.

c. *Soil modification*. The drilling mud is mixed with the in situ soil along the drilled path. This aids in the pipeline installation process by reducing the shear strength of the soil.

d. *Spoil transportation*. Spoils consisting of drilled soil or rock are suspended in the drilling fluid. Fluid returns flowing in the annulus between the drill pipe and the excavated hole are then brought to the surface in the return flow.

e. *Friction reduction*. The lubricating properties of the drilling mud reduce friction between the pipe and the wall of the hole.

f. *Hole stabilization*. The drilling fluid acts to stabilize the hole by providing a mud cake on the wall of the bore and a positive pressure head in the hole. The wall cake helps to seal pores and form a bridging mechanism.

g. *Cleaning and cooling of cutters*. The high-velocity drilling fluid removes the buildup of drilled spoil on bits or reamers. The fluid also dissipates heat from cutters and motors.

An ideal drilling fluid for use in HDD is a bentonite-based freshwater slurry, also referred to as "spud-mud." Bentonite is a naturally occurring swelling clay, also known as montmorillonite. There are many grades and qualities of bentonite commercially available; however, the most commonly used in directional drilling is high-yield sodium montmorillonite mined in Wyoming. Bentonite provides the required low density, viscosity, filtration/wall cake, and gel strength with a minimum of solids production. The principal properties of drilling fluids are outlined below:

a. *Density*. The density of a fluid is a primary factor in determining downhole pressure.

b. *Viscosity*. The viscosity, determined by dividing the shear stress by the shear rate, is a measure of a fluid's resistance to shear.

c. *Funnel viscosity*. This parameter provides a field indication of the relative change in the drilling fluid. It is commonly measured with a Marsh Funnel and involves measuring the time required for a standard volume of mud to drain from a funnel of standard dimension.

d. *Yield point*. The yield point is a measure of the gel strength of the drilling fluid under dynamic conditions. It represents the stress required to initiate flow in a Bingham plastic fluid.

e. Gel strength. The strength of the drilling fluid under static conditions is represented by the gel strength. This measurement is the strength required to break the gel structure.

f. Filtration. Filtration is the tendency for a drilling fluid to lose water into a permeable formation and leave a filter cake on the hole wall.

g. Lubricity. Lubricity is defined as the capacity to reduce friction and is expressed as a coefficient of friction.

The drilling mud used on this project was PARGEL-220. This material is a Wyoming sodium bentonite specially formulated for horizontal drilling where a rapid yield response is needed. PARGEL-220 yields 220 barrels of fluid per metric tonne (ton) of bentonite. This drilling material has been approved for use by the National Sanitation Foundation (NSF) under NSF Standard 60 for Drinking Water Chemicals - Health Effects.

To control the design viscosity, PARGEL-220 was mixed at a ratio of 3.6 to 9.1 kilograms (8 to 20 lb) of bentonite to each barrel of fluid. A dye tracer was added to the drilling fluid to ensure that the drilling fluid could be visually identified in the bore holes at the conclusion of the drilling phase of the project. The dye tracer was Intracid Rhodamine WT Liquid manufactured by Crompton and Knowles of Connecticut. The dye was a very dark red solution generally used for water tracing by visual or fluorometry methods. Mixed with the bentonite-based drilling fluid, it produced a pink tint that was easily identified during the drilling and in the postconstruction investigation.

4 Bore 1

Drilling Details

Drilling began 29 July 1996 with the launch of the downhole assembly. Figure 6 shows a close-up of the drill rig used for all three bores. The cutting edge was a combination of tri-cone roller bits and fluid cutting nozzles. Pipe number 2 was connected and pushed in 3.5 min. This, and all other pipe joints, were approximately 9.5 m (31 ft) in length. Joint number 3 was connected, and drilling operations were stopped for the day. Drilling resumed 30 July, 7:50 a.m., and the remaining 13 joints for the completion of the bore were pushed. Pilot bore completion or "punch out" occurred at 1:32 p.m.

Drilling Fluid Details

For the first bore, the drilling fluid viscosity at the suction was maintained at approximately 58.6 m^3 (62 qt) over the length of the bore. Figure 7 shows the mud pumps and circulation system used on the project. The viscosity at the return was 56.8 m^3 (60 qt). The drilling fluid weight averaged 0.95 kg/ℓ (8.9 lb/gal) for the suction and 1.06 kg/ℓ (9.9 lb/gal) for the return. The sand content averaged 4 percent in the suction and 20 percent in the return.

During the reaming portion of Bore 1, the drilling fluid viscosity in the suction averaged 52.5 m^3 (55.5 qt) and in the return averaged 52.8 m^3 (55.8 qt). The weight of the drilling fluid averaged 0.98 kg/ℓ (9.2 lb/gal) at the suction and 1.03 kg/ℓ (9.6 lb/gal) in the return. The sand content was 7.8 percent in the suction and 13.8 percent in the return.

Table 1 details the mud weight, viscosity, and sand content of the drilling fluid on all three bores. Figure 8 shows a comparison of the drilling mud weights measured at the suction and the return for all three bores. Figures 9 and 10 show the drilling fluid viscosity and sand content, respectively, at the suction and return for each bore. The return viscosity values were impacted by the influx of groundwater in the bore and at the entry drill pit.

Table 1 Drilling Fluid Characteristics of Bores 1 through 3						
	Drilling Fluid kg/ℓ (ppg)		Viscosity ℓ (qt)		Sand Content % by weight	
Operation	Suction	Return	Suction	Return	Suction	Return
Pilot Bore 1	0.95 (8.9)	1.1 (9.9)	58.8 (62.2)	56.8 (60.0)	4.0	20.0
Ream Bore 1	0.98 (9.2)	1.0 (9.6)	52.5 (55.5)	52.8 (55.8)	7.8	13.8
Pilot Bore 2	0.95 (8.9)	1.2 (11.3)	55.8 (59.0)	51.2 (48.4)	5.8	31.2
Ream Bore 2	1.02 (9.5)	1.2 (11.1)	43.5 (46.0)	40.1 (42.4)	6.0	19.6
Pilot Bore 3	0.95 (8.9)	1.0 (9.4)	31.6 (33.4)	30.6 (32.3)	3.2	8.0

As-Built Location

The locations of all three bores were mapped using the Tensor Magnetic Guidance System (MGS) with TruTracker®. The TruTracker® system uses an energized surface coil to communicate with the MGS downhole probe and provides real-time, independent verification of the downhole probe's horizontal direction (azimuth) and vertical position, elevation, or depth. The combined system consisted of:

a. The downhole magnetic guidance probe and interface.

b. The computer, printer, and software.

c. Current control box, wire for surface coil, and DC power source.

The manufacturer claims that the TruTracker® system is capable of ±2 percent depth accuracy and that it can be used to depths of 30.5 m (100 ft) or more. It can be used near ferrous metals and provides immediate records. The as-built drawings for this project were prepared using the data from the MGS and TruTracker® systems.

Figure 11 shows a plan view comparing the planned and actual alignment of Bore 1. Figure 12 shows a profile view of Pilot Bore 1, detailing the planned location, as-built location, and piezometer locations.

The maximum depth of the bore, below the natural ground at the entry point, occurred at the center line of the levee and was 7 m (23 ft) deep. The bore depth below the riverside toe of the levee was 6.6 m (22 ft). At the landside toe of the levee, the depth was 4.8 m (16 ft). A profile of the bore location is shown in Figure 12. The maximum lateral deviation from the planned horizontal alignment was 1.2 m (4 ft).

Internal/External Pressures - Pilot Bore 1

Internal and external drilling fluid pressures were measured during the execution of Bore 1. Internal drilling fluid pressures were measured in the drill stem prior to the exit nozzles. A special pressure sensing device, manufactured by ENSCO and supplied by Sharewell, was used to measure the pressure in the annular space, 0.305 m (1 ft) behind the nozzle. Figure 13 shows the pressure sensing device.

Figure 14 displays the internal and external pressures measured during Pilot Bore 1. Over the length of Pilot Bore 1, the average internal pressure was 1,758 kN/m^2 (255 psi). The bore was analyzed by dividing it into three distinct zones. Internal drilling pressures were held relatively constant over the first zone which included the first 67.5 m (225 ft) of the bore, from STA 1+00 to 3+25. The internal pressure in Zone 1 averaged 1,592.5 kN/m^2 (231 psi) with an average absolute deviation of 75.8 kN/m^2 (11 psi). The second zone, 46.5 m (155 ft) long from STA 3+25 to 4+80, spanned the interval from 7.8 m (26 ft) riverside of the levee center line to the location of Piezometer 2. In this zone, internal pressures were gradually increased until they reached as high as 2,372 kN/m^2 (344 psi). The average pressure was 2,020 kN/m^2 (293 psi) with an average absolute deviation of 138 kN/m^2 (20 psi), marking an average pressure increase of 20 percent from Zone 1 to Zone 2. The internal pressures were then decreased over the third zone which extended from STA 4+80 to the end of the pilot bore at STA 6+32. In Zone 3 the average internal pressure was 1,634 kN/m^2 (237 psi) with an average absolute deviation of 34.5 kN/m^2 (5 psi).

External drilling pressures were measured throughout the length of the drive using a pressure transducer located 0.305 m (1 ft) behind the nozzle on the pilot bit. The external pressures were a small fraction of the internal pressures at every measurement location. In Zone 1, the average external pressure was 324 kN/m^2 (47 psi). In Zone 2, the average external pressure was 345 kN/m^2 (50 psi). For Zone 3, the average external pressure was 330.9 kN/m^2 (48 psi). This range of pressures (324 to 345 kN/m^2 (47 to 50 psi)) throughout the entire bore was very small, regardless of internal pressure.

Piezometer Readings - Pilot Bore 1

Table 2 presents a summary of the pressure readings recorded during Pilot Bore 1 at the six piezometer locations.

As the drill stem passed within 3 m (10 ft) of Piezometer 1, the largest increase in pressure of 1.4 kN/m^2 (0.5 psi) was recorded. Figure 15 shows the pressure at Piezometer 1 as a function of time. The pressure spike shown on Figure 15 corresponds to the time when the drill passed the piezometer. The duration of the pressure increase was very short, lasting approximately 30 min.

Table 2
Piezometer Reading During Pilot Bore 1

Piez. No.	Average P_i kN/m^2 (psi)	Average P_e kN/m^2 (psi)	P_e/P_i %	Minimum Distance From Piezometer m (ft)	Maximum Change in ΔP kN/m^2 (psi)	$\Delta P/P_i$ %	$\Delta P/P_e$ %
1	1,593 (231)	331 (48)	21	3.0 (10)	3.4 (0.5)	0.2	1.0
2	1,593 (231)	331 (48)	21	9.6 (32)	1.4 (0.2)	0.08	0.4
3	2,020 (293)	345 (50)	17	6.6 (22)	0	0	0
4	2,020 (293)	345 (50)	17	12.9 (43)	0	0	0
5	1,634 (237)	324 (47)	20	7.2 (24)	0.7 (0.1)	0.04	0.2
6	1,634 (237)	324 (47)	20	14.7 (49)	0.7 (0.1)	0.04	0.2

Note: P_i = internal drillstem pressure; P_e = external annulus pressure; ΔP = piezometer pressure.

The excess pressure caused by the drilling then dissipated until pressures returned to the initial reading. A small pressure increase was recorded on Piezometer 1 at approximately 12:20 p.m. This increase corresponds to the drilling of a subsequent pipe section, after the cutting mechanism had passed the piezometer.

As the cutting mechanism passed Piezometer 2, pressure increases were observed on the order of 1.4 kN/m^2 (0.2 psi), as compared with 3.4 kN/m^2 (0.5 psi) in Piezometer 1. Figure 16 shows the pressure in Piezometer 2 as a function of time. The cutting mechanism passed Piezometer 2 at a minimum distance that was over three times larger than the distance for Piezometer 1 (9.6 m (32 ft) as compared to 3 m (10 ft)). However, unlike the readings in Piezometer 1, the drilling pressure did not dissipate as quickly. Pressure increased 1.4 kN/m^2 (0.2 psi) as the cutting mechanism passed the piezometer and then decreased to a pressure that was still elevated 0.7 kN/m^2 (0.1 psi) higher than the initial reading. The pressure measured at the piezometer remained elevated by 0.7 kN/m^2 (0.1 psi) for 2 hr after the cutting mechanism had passed when readings were discontinued.

Piezometers 3 and 4, which were located at the crest of the levee and 4.5 m (15 ft) below the elevation of the bore, experienced no pressure changes while the cutting mechanism passed the piezometers. This is not surprising, since the pressure increases measured at the other piezometer locations were relatively small. Since the water pressure acting on the drill bit is approximately 3.4 kN/m^2 (0.5 psi) per foot of groundwater, the pressure changes induced by the cutting mechanism would have to be greater than 51.7 kN/m^2 (7.5 psi) for changes in pressure to register at the piezometer location. All of the pressures measured on the test were well below this level.

As the cutting mechanism passed Piezometer 5 at a minimum distance of 7.2 m (24 ft), pressure increases of 0.7 kN/m^2 (0.1 psi) were recorded over a 30-min time period. Subsequent pressure increases were not recorded at this location. Figure 17 shows the pressure at Piezometer 5 as a function of time.

Figure 18 shows the pressure at Piezometer 6, which the cutting mechanism passed at a distance of 14.7 m (49 ft). The drilling mechanism passed the piezometer location at 9:45 a.m. Pressure then decreased to a "baseline" level that was 0.7 kN/m^2 (0.1 psi) below the initial reading. It is thought that the pressure induced by the cutting mechanism is 0.7 kN/m^2 (0.1 psi) but the reading was not taken at the piezometer until after the initial increase had occurred. The pressure then dissipated to the baseline level.

When examining Table 2, it is interesting to note that the pressure increase in Piezometers 5 and 6 was the same, at 0.7 kN/m^2 (0.1 psi), even though there was a 7.5-m (25-ft) difference between their distances from the bore. This could be due to the accuracy limitations of the pneumatic piezometer pressure recorders as the minimum readable pressure change was 0.7 kN/m^2 (0.1 psi). Therefore, the pressures may have been slightly different but not within the accuracy of the instrument.

Reaming Bore 1

A 50.8-cm (20-in.) fly cutter was used during the reaming and pullback of Bore 1. Figure 19 shows the fly cutter reaming assembly. The fly cutter reamed the pilot bore while at the same time installing a 30.5-cm- (12-in.-) diam steel pipe. Figure 20 shows the drill stem, fly cutter, and pipeline prior to installation of the pipe. Figure 21 shows the installation of the pipeline.

The reaming process began at 1:10 p.m., 1 August 1996. The entire operation was complete by 8:05 p.m. The average advance rate was 27 m/hr (90 ft/hr). No inadvertent returns were noted at any location along the bore.

Internal/External Pressures - Reaming Bore 1

During the reaming portion of Bore 1, the average internal pressure measured 910 kN/m^2 (132 psi) with an average absolute deviation of 186 kN/m^2 (27 psi). It is important to note that average internal pressures during the reaming portion of the drive were only 52 percent of those recorded during the pilot bore. External pressures during the reaming drive averaged 372 kN/m^2 (54 psi) with an average absolute deviation of 43.4 kN/m^2 (6.3 psi). External pressures represented 41 percent of the internal pressures during the reaming portion of the bore. It should be noted that the average external pressures for both the pilot and reaming bores were approximately equal, even though the internal pressures were nearly twice as high on the pilot bore.

Piezometer Readings - Reaming and Pullback Bore 1

The pressure changes measured at the piezometer locations were dramatically different for the reaming process than for the drilling of the pilot bore. Unlike the pressure spikes that were observed in the pilot drilling, pressures during the reaming and pullback operations fluctuated with every pulled drill stem section after the reamer passed the piezometer.

Table 3
Piezometer Reading During Reaming of Bore 1

Piezometer No.	Average Internal Pressure, P_i kN/m^2 (psi)	Average External Pressure, P_e kN/m^2 (psi)	P_e/P_i %	Minimum Distance m (ft)	Pressure Fluctuations kN/m^2 (psi)
1	1,103 (160)	296 (43)	27	3 (10)	1.4 - 4.8 (0.2 - 0.7)
2	1,103 (160)	296 (43)	27	9.6 (32)	1.4 (0.2)
3	993 (144)	400 (58)	40	6.6 (22)	0 (0)
4	993 (144)	400 (58)	40	12.9 (43)	0 (0)
5	655 (95)	393 (57)	60	7.2 (24)	0.7 (0.1)
6	655 (95)	393 (57)	60	14.7 (49)	0 (0)

Figure 22 shows the pressure at Piezometer 1 as a function of time. When the reaming assembly passed the piezometer at a distance of 3 m (10 ft), pressure fluctuations were on the order of 1.4 kN/m^2 (0.2 psi). After the initial increase in pressure, subsequent pressure fluctuations correspond to times when the drill rig was pulling back a drill stem segment. Pressure stabilization corresponds to times when the drilling mud was not circulating while sections of the drill stem were being removed. After the reaming assembly had passed the piezometer by 37 m (124 ft) (four drill stem segments) the changes in pressure increased to 2 kN/m^2 (0.3 psi). The pressure fluctuations continually increased until they reached a maximum of 4.8 kN/m^2 (0.7 psi) at the end of the reaming and pullback operations. These changes in pressure are indicative of the pressure required to force the cuttings back to the mud pit located at the reamer entry point closest to Piezometer 1.

Pressure changes in Piezometer 2, located 9.6 m (32 ft) from the bore path, were much less pronounced than in Piezometer 1, with pressure fluctuations measuring 1.4 kN/m^2 (0.2 psi) as the reamer passed the piezometer. Figure 23 shows the pressure in Piezometer 2 as a function of time. Monitoring of Piezometer 2 was aborted due to the fact that only two piezometer readers were located on site, and the reamer was approaching Piezometer 5 where pressure

measurements could have been more significant due to the proximity of the piezometer to the bore path.

Pressure changes in Piezometer 5, located 7.2 m (24 ft) from the bore path were 0.7 kN/m^2 (0.1 psi) and were recorded as a single pressure spike. Pressures quickly decreased to the initial baseline reading after the reaming assembly passed the piezometer location. Figure 24 shows the changes in pressure as a function of time measured at Piezometer 5 during the reaming and pullback operations.

5 Bore 2

Drilling Details

Drilling for Pilot Bore 2 began at 8:58 a.m., 6 August 1996. The drilling was completed on the same day with punchout at 1:51 p.m. The drilling progressed very quickly with eight of the 9.3-m (31-ft) joints pushed in under 3 min each. Due to logistical problems and the pace of the drilling, exact drilling rates were not recorded for every joint; however, Table 4 shows the times recorded for the drilling of 10 joints during Pilot Bore 2.

Table 4
Time Required to Drill Pilot Bore 2 Pipe Sections

Joint No.	Time to Push 9.5-m (31-ft) Pipe Joint
2	4 min
3	2 min 54 sec
4	1 min 7 sec
5	3 min
7	2 min 11 sec
8	1 min 50 sec
9	1 min 30 sec
12	2 min 37 sec
13	2 min 32 sec
14	1 min 26 sec

Inadvertent returns were noticed 13.2 m (44 ft) from the exit of the drill stem while pushing joint 15. (Punchout occurred during joint 16.) The nature of the returns was very similar to that observed during Pilot Bore 1; the fluid oozed to the surface above the path of the bore, as can be seen in Figure 25. Figure 26 shows punchout of the pilot bore on the landside of the levee.

Drilling Fluid Details

For Pilot Bore 2, the drilling fluid viscosity at the suction averaged 56 ℓ (59 qt). At the return, the average fluid viscosity was 48.4 ℓ (51.2 qt). The drilling fluid weight averaged 0.95 kg/ℓ (8.9 lb/gal) at the suction and 1.2 kg/ℓ (11.3 lb/gal) at the return. The sand content of the drilling fluid was 5.8 percent at the suction and 31.2 percent at the return. Details of the drilling fluid for all three bores can be found in Figures 8 through 10.

During the reaming of Bore 2, the fluid viscosity at the suction was 44 ℓ (46 qt) and 40 ℓ (42.2 qt) at the return. The weight of the drilling fluid averaged 1.0 kg/ℓ (9.5 lb/gal) in the suction and 1.2 kg/ℓ (11.1 lb/gal) in the return. The sand content averaged 6 percent in the suction and 19.6 percent in the return.

As-Built Location

Figure 27 shows a plan view with the planned and as-built location of Pilot Bore 2. Figure 28 shows a profile of Pilot Bore 2. The maximum depth of the bore occurred at the center line of the levee and was 6.6 m (22 ft) below the original grade. The bore depth below the riverside toe of the levee was 6.3 m (21 ft). At the landside toe of the levee, the depth was 5.4 m (18 ft) below the original ground elevation. A profile of the bore location is shown in Figure 28. In the area of the piezometers, the maximum lateral deviation along the alignment was ±0.3 m (±1 ft).

Internal/External Pressures - Pilot Bore 2

Internal drilling pressures varied greatly during Pilot Bore 2, with a minimum pressure of 689 kN/m^2 (100 psi) and a maximum pressure of 1,724 kN/m^2 (250 psi). Table 5 separates the bore into three zones, as with the first pilot bore, and compares the internal drilling pressures and deviations.

Table 5
Internal Drilling Fluid Pressures on Pilot Bores 1 and 2

	Pilot Bore 1		Pilot Bore 2	
Zone	Average Pressure kN/m^2 (psi)	Average Absolute Deviation kN/m^2 (psi)	Average Pressure kN/m^2 (psi)	Average Absolute Deviation kN/m^2 (psi)
Zone 1 STA 100 - 325	1,593 (231)	76 (11)	1,213 (176)	276 (40)
Zone 2 STA 326 - 452	2,020 (293)	138 (20)	1,241 (180)	131 (19)
Zone 3 STA 453 - 632	1,634 (237)	34 (5)	1,041 (151)	4.8 (0.7)
Average	1,758 (255)	-	1,213 (176)	-

It is important to note that in Zone 3 only two pressure readings were recorded due to a problem with the instrumentation. This should be considered as a reason for the low average absolute deviation. As can be seen from Table 5, internal pressures recorded during Pilot Bore 2 were substantially lower than those recorded during Pilot Bore 1. Figure 29 shows the fluctuations in internal and external pressures during Pilot Bore 2. When comparing the internal pressures on Pilot Bores 1 and 2, the average internal pressure for Bore 1 was 1,758 kN/m^2 (255 psi), as compared to the average internal pressure for Bore 2 of 1,213 kN/m^2 (176 psi).

External pressures recorded on Pilot Bore 2 averaged 29 percent of the recorded internal pressures. The average absolute deviation was 4 percent. As with Bore 1, external pressures were relatively constant at 345 kN/m^2 (50 psi) throughout Bore 2, regardless of the internal pressures.

Piezometer Readings - Pilot Bore 2

Table 6 shows a summary of the pressure readings recorded during Pilot Bore 2.

Table 6
Piezometer Readings During Pilot Bore 2

Piez. No.	Average Internal Drillstem Pres. P_i kN/m^2 (psi)	Average External Annulus Pres. P_e kN/m^2 (psi)	P_e/P_i %	Min. Dist. From Piez. m (ft)	Max. Change in Piez. Pres. ΔP kN/m^2 (psi)	$\Delta P/P_i$ %	$\Delta P/P_e$ %
1	1,213 (176)	324 (47)	27	6.6 (22)	0.7 (0.1)	0.1	0.2
2	1,213 (176)	324 (47)	27	2.7 (9)	2.1 (0.3)	0.2	0.6
3	1,241 (180)	358 (52)	29	5.4 (18)	0 (0)	0	0
4	1,241 (180)	358 (52)	29	5.7 (19)	0 (0)	0	0
5	1,041 (151)	345 (50)	33	1.5 (5)	2.8 (0.4)	0.3	0.8
6	1,041 (151)	345 (50)	33	6.6 (22)	4.2 (0.6)	0.4	1.2

The cutting mechanism passed Piezometer 1 at a distance of 6.6 m (22 ft). Figure 30 shows the pressure in Piezometer 1 as a function of time. As the cutting mechanism passed the piezometer, the pressure increased by 0.7 kN/m^2 (0.1 psi) for less than 15 min. A subsequent pressure spike at approximately 30 min after the initial pressure increase corresponds to the drilling of another pipe section.

The maximum change in pressure measured in Piezometer 2 was 2.1 kN/m^2 (0.3 psi). Figure 31 shows the pressure in Piezometer 2 as a function of time. The cutting mechanism passed Piezometer 2 at a distance of 2.7 m (9 ft). The maximum pressure was observed shortly after the cutting mechanism passed this minimum distance. Within 30 min the pressure had dissipated to within 0.7 kN/m^2 (0.1 psi) of the initial reading.

Like Bore 1, Piezometers 3 and 4 experienced no pressure changes while the cutting mechanism passed them. Again, this is due to the depth (4.5 m (15 ft)) beneath the bore holes at which the piezometers were placed. The pressure created by drilling pipe sections was not high enough to create readings at these two piezometers.

Piezometer 5 was within 1.5 m (5 ft) of the bore path, the nearest the cutting mechanism came to any piezometer during the entire project. The maximum pressure change recorded by Piezometer 5 was 2.8 kN/m^2 (0.4 psi). Figure 32 shows the Piezometer 5 pressure as a function of time. The pressure becomes highest as the piezometer is passed by the cutting mechanism. The pressure then dissipates until the next pipe section is pushed.

Although Piezometer 5 was the closest to the bore path, the highest pressure readings were observed at Piezometer 6. Figure 33 shows the pressure at Piezometer 6 as a function of time. The cutting mechanism passed Piezometer 6 at a distance of 6.6 m (22 ft), the largest distance for any piezometer during Pilot Bore 2. To explain this seemingly disjunct phenomenon it is important to focus on the drilling procedure at this location. As the cutting mechanism passed Piezometer 6, the driller was experiencing difficulty maintaining planned alignment. As a result, the drill stem was retracted and redrilled four consecutive times. The redrilling process caused pressure spikes that increased in magnitude with each consecutive pass. It is believed that the redrilling caused the pressure buildup that contributed to the highest pressure readings at Piezometer 6. These pressure changes lasted over a period of 30 min. The pressure then returned to the initial reading before increasing again as the next pipe section was drilled.

Reaming Bore 2

A barrel reamer was used to ream Pilot Bore 2. However, no pipe was pulled back during the reaming process. Figure 34 shows the barrel reamer prior to the reaming process. Figure 35 shows the barrel reamer, connected in series behind the fly cutter, at the completion of the reaming process of Bore 2.

Reaming operations for Bore 2 began at 9:35 a.m., 7 August 1996, and were completed at 12:10 p.m. on the same day. The reaming process was completed very quickly, but was slightly slower than the pilot bore. Table 7 shows the times recorded for selected pipe joints during the reaming operation.

Table 7 Time Required to Ream Bore 2	
Joint No.	Reaming Time per 9.3-m (31-ft) Joint
5	3 min 21 sec
6	5 min 23 sec
8	2 min 46 sec
10	3 min 29 sec
11	2 min 7 sec
12	2 min 30 sec
13	2 min 33 sec
14	3 min 23 sec

Internal/External Pressures - Reaming Bore 2

Unfortunately, due to a malfunction with the pressure recording apparatus, internal and external pressures were recorded only at the beginning of the reaming of Bore 2, over the first (47 m) 157 ft. Pressures were recorded from the time the reaming assembly passed Piezometers 1 and 2 until the end of the reaming process. A total of seven readings were taken with the pressure recording apparatus. Over that range, the internal pressure averaged 565 kN/m^2 (82 psi), with an average absolute deviation of 87 kN/m^2 (12.6 psi). It should also be pointed out that, even though internal pressures varied significantly for all three bores, the external pressures remained within a relatively narrow range, between 324 and 358 kN/m^2 (47 and 52 psi) for both the pilot bore and reaming processes.

Piezometer Readings - Reaming Bore 2

Table 8 shows a summary of the pressure readings recorded during the reaming of Bore 2. As previously described, the reaming operations were performed with a barrel reamer; however, no pipe was pulled into the reamed bore. In addition, internal and external pressures were measured only in Zone 3, associated with Piezometers 1 and 2.

During the reaming of Bore 2, Piezometers 1 and 2 exhibited pressure spikes unlike the readings recorded during the reaming of Pilot Bore 1, which displayed a series of pressure fluctuations during the reaming process. This is likely due to the fact that a pipe was not pulled into the hole. The pressure spikes were observed as the barrel reamer passed the piezometers, and no further pressure increases were measured. However, Piezometers 5 and 6 experienced more pressure fluctuation, similar to Piezometer 2 during reaming of Bore 1. The initial pressure spikes in Piezometers 5 and 6 are due to the passage of the barrel reamer. Subsequent pressure fluctuations are due to mud pressures required to circulate

Table 8
Piezometer Reading During Reaming of Bore 2

Piezometer No.	Average Internal Pressure, P_i kN/m^2 (psi)	Average External Pressure, P_e kN/m^2 (psi)	P_e/P_i %	Minimum Distance m (ft)	Pressure Fluctuations kN/m^2 (psi)
1	565 (82)	324 (47)	57	6.6 (22)	0.7 (0.1)
2	565 (82)	324 (47)	57	2.7 (9)	0.7 (0.1)
3	-	-	-	5.4 (18)	0 (0)
4	-	-	-	5.7 (19)	0 (0)
5	-	-	-	1.5 (5)	0.7 - 1.4 (0.1 - 0.2)
6	-	-	-	6.6 (22)	0.7 - 2.1 (0.1 - 0.3)

the mud back to the pit located on the landside of the levee. These subsequent pressure fluctuations were not seen at Piezometers 1 and 2 because of their close proximity to the mud pit where minimal pressure was required to circulate the mud.

Figure 36 shows the pressure measured in Piezometer 1 versus time. The reaming assembly passed Piezometer 1 at a distance of 6.6 m (22 ft). When the reaming assembly passed the piezometer, a pressure increase of 0.7 kN/m^2 (0.1 psi) was observed. This increase lasted less than 30 min.

Piezometer 2 experienced pressure changes very similar to those of Piezometer 1. Figure 37 shows the pressure at Piezometer 2 as a function of time. The reaming assembly passed Piezometer 2 with a distance of 2.7 m (9 ft). A short pressure increase of 0.7 kN/m^2 (0.1 psi) occurred soon after the reaming assembly passed the piezometer. The duration of the pressure increase was less than 15 min.

The pressure observed in Piezometer 5 had more variation than in Piezometers 1 and 2. Figure 38 shows the pressure in Piezometer 5 as a function of time. When the reaming assembly passed the Piezometer (at a distance of 1.5 m (5 ft)), the pressure began to fluctuate. The highest fluctuation was approximately 1.4 kN/m^2 (0.2 psi). Once the reamer had passed, the pressure returned to the level it held before the reaming process began.

Figure 39 shows the pressure measured in Piezometer 6 during the reaming of Bore 2. Although the reaming assembly passed the other piezometers at a closer distance, Piezometer 6 (passed at 6.6 m (22 ft)) recorded the maximum pressure fluctuations. These fluctuations were as large as 2.1 kN/m^2 (0.3 psi). The subsequent pressure fluctuations correspond to the reaming and pulling of successive drillstem sections.

6 Bore 3

Bore 3 consisted of only a pilot bore. No reaming processes took place, as the pilot bore was terminated prior to punchout of the pilot bore.

Drilling Details

Drilling for the pilot bore began at 8:56 a.m., 8 August 1996, and was completed at 1:26 p.m. when Joint 13 was drilled. As with Pilot Bore 3, the drilling progressed very rapidly with an average advance rate of 27 m/hr (90 ft/hr). Table 9 displays some recorded drilling times for 9.3-m (31-ft) joint drillstem segments.

Table 9
Time Required to Drill Pilot Bore 3

Joint No.	Drilling Time for 9.3-m (31-ft) Joint
2	1 min 35 sec
3	1 min 1 sec
4	1 min 6 sec
5	2 min 40 sec
6	1 min 27 sec
8	1 min 24 sec
9	1 min 50 sec
10	1 min 32 sec
11	1 min 42 sec

Drilling Fluid Details

The drilling mud used for the Pilot Bore 3 was much different than that used for the first two pilot bores. The fluid viscosity at the suction was 31.6 ℓ (33.4 qt) and 30.6 ℓ (32.3 qt) at the return, much lower than for Bores 1 and 2. Mud weight averaged 8.4 kg/ℓ (8.9 lb/gal) at the suction and 8.9 kg/ℓ (9.4 lb/gal) at the return.

The sand content of the drilling mud was 3.2 percent at the suction and 8 percent at the return. Details of the drilling mud can be found in Figures 8 through 10.

As-Built Location

Pilot Bore 3 was launched from the same location as Pilot Bore 2; however, the planned routes of the pipeline were markedly different. As can be seen in the plan view of the bore in Figure 5, the planned alignment was in a horizontal curve and was to cross the levee on a path parallel to the other two bores. The pilot bore crossed under the riverside toe of the levee 6.0 m (20 ft) below the ground elevation. On the land side of the levee, the pilot bore was 5.4 m (18 ft) below the toe. Figure 40 shows a plan view of the planned and as-built location of Pilot Bore 3. Figure 41 shows the profile of Bore 3 alignment.

Internal/External Pressures - Pilot Bore 3

Figure 42 shows the internal and external pressures recorded during Pilot Bore 3. As can be seen from the figure, internal pressures were held relatively constant during the pilot bore but increased markedly at the location of Piezometers 1 and 2. The magnitude of the external pressures was relatively constant, 324 to 352 kN/m^2 (47 to 51 psi), and within the same narrow range measured on Bores 1 and 2, even though the external pressures varied over a much wider range.

Piezometer Readings - Pilot Bore 3

Table 10 shows a summary of the pressure readings recorded during Pilot Bore 3.

Figure 43 shows the pressure measured in Piezometer 1 as a function of time. The cutting mechanism passed Piezometer 1 at a distance of 13.2 m (44 ft). Pressure fluctuations before passing the Piezometer are due to the bore being redrilled for position. The maximum pressure change was approximately 1.4 kN/m^2 (0.2 psi).

The cutting mechanism passed Piezometer 2 at a distance of 6.0 m (20 ft), as compared to 13.2 m (44 ft) for Piezometer 1. Due to the proximity of the cutting mechanism, it is not surprising that the maximum pressure change of 0.7 kN/m^2 (1.0 psi) at Piezometer 2 was much higher than that of Piezometer 1. Figure 44 shows the pressure at Piezometer 2 as a function of time. Again, the pressure fluctuations before the cutting mechanism passed the Piezometer correspond to redrilling for position and are indicative of a pressure buildup caused by the redrilling.

Table 10
Piezometer Reading During Pilot Bore 3

Piez. No.	Average Internal Drillstem Pres. P_i kN/m² (psi)	Average External Annulus Pres. P_e kN/m² (psi)	P_e/P_i %	Min. Dist. From Piez. m (ft)	Max. Change in Piez. Pres. ΔP kN/m² (psi)	$\Delta P/P_i$ %	$\Delta P/P_e$ %
1	1,200 (174)	352 (51)	29	13.2 (44)	1.4 (0.2)	0.1	0.4
2	1,200 (174)	352 (51)	29	6.0 (20)	0.7 (1.0)	0.6	2
3	1,213 (176)	352 (51)	29	10.8 (36)	0 (0)	0	0
4	1,213 (176)	352 (51)	29	3.9 (13)	0 (0)	0	0
5	2,116 (307)	324 (47)	15	6.3 (21)	2.8 (0.4)	0.1	0.85
6	2,116 (307)	324 (47)	15	1.5 (5)	32.4 (4.7)	1.5	10

Figure 45 shows the pressure measured in Piezometer 5 as a function of time. Piezometer 5 was passed at a distance of 6.3 m (21 ft), less than one-half the distance for Piezometer 1. Thus the maximum pressure increase of 2.8 kN/m² (0.4 psi) (twice that of Piezometer 1) appears reasonable.

The cutting mechanism passed Piezometer 6 at a distance of 1.5 m (5 ft), the smallest distance for any piezometer during Pilot Bore 3. Figure 46 shows the pressure at Piezometer 6 as a function of time. Since the cutting mechanism was very close to Piezometer 6, it is not surprising that this piezometer recorded the highest pressure change, 32.4 kN/m² (4.7 psi). While this pressure change seems to be high, it corresponds to only 10 percent of the average external pressure measured in the annular space of the bore of Zone 1 during Pilot Bore 3.

7 Autopsy

General Description

Upon completion of the three bores, an autopsy was conducted by excavating from the surface to expose the bores and make visual inspection of the borepaths. The dye tracer added to the drilling fluid allowed visual identification of any drilling fluid migration around the bores. Figure 47 shows the track-hoe excavating on the landside of the levee. Excavation took place with a Komatsu PC 200 track-hoe to expose the bores above the water table. The bores were easily found and exposed on the landside of the levee above the water table as seen in Figure 48. Excavation on the landside of the levee exposed only Bores 1 and 2 due to the early termination of Pilot Bore 3. On the riverside of the levee, Bores 1, 2, and 3 were exposed in the autopsy.

Excavation below the water table made identification of the bore hole very difficult due to the groundwater and drilling fluid migration into the excavation, as can be seen in Figure 49. To aid in excavation below the water table, trench boxes (Figure 50) were used to provide temporary stability to the excavation and protect workers. However, bore hole identification was still difficult due to the presence of groundwater, as seen in Figure 51. As a result, the approximate bore hole location was found by excavating through the bore and looking for dye tracer, shown in Figure 52.

Bore 1

Landside - Bore 1

An attempt to expose Bore 1 was made at a location 3 m (10 ft) from the exit point; however, the hole collapsed, probably due to the weight of the track-hoe. Drilling fluid ran from the bottom of the bore hole into the excavation made by the track-hoe. There was a well-defined mud cake approximately 3 cm (1.2 in.) around the collapsed bore.

At a location approximately 4.5 m (15 ft) from the exit of Bore 1, the hole was exposed and remained stable. Figure 53 shows that the hole was well-defined but not as round as Bore 2. This is attributed to the fact that a fly cutter was used on

Bore 1, whereas a barrel reamer was used on Bore 2, making Bore 2 much more uniform in shape. The open bore measured 0.5 m (1.7 ft) in diameter. The bore was approximately one-fourth full of drilling fluid.

Hydrofracture was observed at this location and manifested as a seam of drilling fluid radiating horizontally from the bore toward the lake. The limit of the hydrofracture was 1.9 m (6.4 ft) from the edge of the bore hole. At the zone where this hydrofracture was visible, there was a thin seam of clay and a mottling of the clay stratum. This can be seen in Figure 54. The clay seam was approximately 7.6 cm (3 in.) thick with iron stains at the top of the clay layers. The drilling mud had protruded into the soil at the interface between the gray clay and the iron-stained soil.

Radial hydrofracture was also observed on the opposite side of the bore hole, extending toward the limits of the excavation, approximately 1.8 m (6 ft), toward Bore 2. The seam of drilling fluid was approximately 1.9 cm (3/4 in.) thick in close proximity to the bore and became thinner as it moved away from the bore. Upon further excavation, it was found that the drilling fluid communicated between Bores 1 and 2.

Bore 1 was further excavated toward the levee in an attempt to view the bore hole below the water table; however, the soil collapsed into the borehole almost immediately upon exposure. The bore was easily identified by the dye tracer in the drilling fluid, and no hydrofracture was observed below the water table.

Riverside - Bore 1

Bore 1 was exposed on the riverside at a depth of 1.8 m (5.9 ft) to the bottom of the bore. The soil above the bore had collapsed and filled in the hole, as shown in Figure 55. The hole eroded to a maximum diameter of 0.8 m (2.7 ft) and had a vertical diameter of 0.3 m (1.1 ft). No signs of hydrofracture were evident. Further excavation revealed a stable bore hole at a depth of 2.1 m (7 ft) that measured 0.5 m (1.7 ft) in diameter (Figure 56).

Bore 2

Landside - Bore 2

Excavation on the landside of the levee began very near the exit point of Bore 2. The excavation began 4.5 m (15 ft) levee side of the exit point. When the bore was exposed, it was found to have collapsed, probably due to the weight of the track-hoe. Measured depth at this location was 1.4 m (4.7 ft) from ground surface to the bottom of the bore hole. It was readily apparent that the drilling fluid had migrated beyond the extent of the bore. The drilling fluid migrated away from the bore path, manifesting in radial cracks ranging from 1.3 to 3.8 cm (0.5 to 1.5 in.) in thickness and migrating up to 0.9 m (3 ft) away from the bore.

The track-hoe was then relocated to excavate at a location 6 m (20 ft) from the exit point of the bore. At this location the bore was stable and well-defined (Figure 57). The hole was round in appearance and was lined with a mud cake approximately 0.6 cm (1/4 in.) thick. The maximum horizontal and vertical diameter of the bore hole measured 0.5 m (1.6 ft). The measured depth from ground surface to the bottom of the bore was 1.8 m (6 ft).

At this excavation location there was a vertical crack, approximately 0.3 cm (1/8 in.) thick, that originated at the bore hole and continued vertically to the ground surface. The exposed vertical crack is shown in Figure 58. It was at this location that inadvertent returns were seen on the ground surface during the drilling of Pilot Bore 2. Excavation continued to follow the bore path, exposing 12 m (40 ft) of the bore in the longitudinal direction, extending toward the levee, until the depth of the bore was approximately 3.3 m (11 ft). Along the 12 m (40 ft) of excavation, the vertical crack was traced along the path of the bore, extending vertically from the bore path to the surface. Figure 59 shows the exposed vertical crack to a depth of 3.3 m (11 ft). Figure 60 is a closeup view of the vertical crack. At surface locations where inadvertent returns were not seen, the drilling mud was confined by a layer of compacted top soil approximately 7.6 cm (3 in.) thick. Upon removal of the top soil, drilling mud was seen to have pooled under the confining top soil. At a bore depth of 3.3 m (11 ft), the water table was encountered. The vertical hydrofracture was no longer visible below the water table, and it appeared that the water table confined the hydrofracture.

Riverside - Bore 2

Bore 2 was exposed on the riverside of the levee and was found to be stable at a depth of 2.7 m (9 ft) from ground surface to the bottom of the bore. Figure 61 shows Bore 2 on the riverside. The hole was uniform with a distinct mud cake around the perimeter. The maximum diameter of the bore was 0.8 m (2.5 ft), and the minimum diameter was 0.5 m (1.5 ft). There were no signs of hydrofracture on the riverside of the levee on Bore 2. As excavation continued, the bore was easily defined and was approximately one-eighth full of drilling mud (Figure 62).

Bore 3

Bore 3 was first excavated very close to the entry point where there was only 0.6 m (1.9 ft) of distance between Bores 2 and 3. The appearance of Bore 3, which was not reamed, was diamond shaped and not well-defined. The observed area was simply a zone, measuring 0.2 m (0.6 ft) on the maximum diameter and 0.1 m (0.3 ft) on the minimum diameter, filled with soft pink mud. Figure 63 shows a zone of soft mud identified as Pilot Bore 3. The edge of the zone was-defined by locating soft zones by hand. A horizontal hydrofracture, approximately 5 cm (2 in.) thick, connected Bores 2 and 3 at this location.

The bore path was further exposed in the direction toward the levee. The measured depth at the new excavation location was 2.3 m (7.8 ft) from the ground surface to the bottom of the bore. At this location the borehole was open and

spiral shaped, measuring 0.2 m (0.8 ft) on the maximum diameter and 0.2 m (0.6 ft) on the minimum diameter. Sight down the borehole was approximately 9 m (30 ft) with the aid of a flashlight.

At an additional excavation location closer to the levee, the depth of Pilot Bore 3 was 3.4 m (11.3 ft). The hole was very irregular in shape, appearing rectangular. Subcavity erosion, appearing as a channel, was seen inside the bore. The channel was very straight, appearing to be cut with a jet on the drilling assembly. The excavation was extended farther toward the levee; however, the borehole was collapsed, probably because it was below the water table, and limited information was gathered.

8 Overview/Comparison of All Test Bores

Piezometer Readings

During the three pilot bores and the two reaming bores, readings were taken at all six piezometer locations. In the 2-month period prior to the field test, readings were taken on an almost daily basis to establish a baseline and to correlate piezometric pressure changes with river stages. The Mississippi River is approximately 1.6 km (1 mile) from the site. The river stages fell by approximately 9 m (30 ft) between the time the piezometers were installed and the HDD bores were completed. The corresponding piezometric fluctuations over this period are shown in Figure 64. The maximum change in piezometric pressures between high-water and low-water (end of construction) river stages was 0.8 m (2.6 ft), or 7.6 kN/m^2 (1.1 psi), in Piezometer 1. By contrast, the maximum increase in piezometric pressures during the first pilot boring operations was 3.4 kN/m^2 (0.5 psi) (45 percent of the change due to river stages) at Piezometer 1, when the drilling head was within 3 m (10 ft) of the piezometer. The impact of duration of changes in pore pressure was much greater for river stage changes than for any of the drilling operations.

No influence of drilling pressures was observed during any of the boring operations in Piezometers 3 and 4 installed through the crest of the levee. The tips of the piezometers were approximately 6 to 7.6 m (20 to 25 ft) below the closest approach of the bores. The small increases in piezometric pressures were very short lived. Typically during pilot bore drilling, the pressure decayed to the hydrostatic level within ½ hr after the drill head passed.

Overview of Internal/External Pressures

Internal and external pressures were measured along the bore paths during drilling. Figure 65 shows the measured internal and external pressures recorded during all three pilot bores and the reaming of Bore 1. The maximum internal pressure was 2,475 kN/m^2 (359 psi) and occurred during Pilot Bore 3. The minimum internal pressure was 462 kN/m^2 (67 psi), recorded during the reaming of Bore 1. Significant variation was observed within this wide range of values. Pilot Bore 1 had three distinct zones where the pressures were systematically held

constant within each zone. Pilot Bore 2 had the most variation of internal pressure, ranging from 689 to 1,724 kN/m^2 (100 to 250 psi). Internal pressure during Pilot Bore 3 was held fairly constant at a pressure of 1,206 kN/m^2 (175 psi) except for one 15-m (50-ft) zone where the pressure was increased to 2,475 kN/m^2 (359 psi). For the reaming of Bore 1, the internal pressures varied over a lower range of pressures, from 462 to 1,034 kN/m^2 (67 to 150 psi).

The external pressure, which is of the utmost concern for levee stability and hydrofracture potential, remained within a very small range of 324 to 358 kN/m^2 (47 to 52 psi) regardless of the internal mud pressures. The high level and variation of the internal pressure did not have an impact on the external pressure. This is largely due to the head losses experienced by the fluid as it exits the drillstem through the cutting nozzle and enters the annular space. The vast majority of fluid pressure in the internal drillstem is dissipated and the external pressure remains constant. This was found to be true over the wide range of internal pressures and the varying mud weights.

Autopsy Findings

The postconstruction autopsy revealed stable boreholes on both the landside and riverside of the levee above the water table. Below the water table, the path of the bore was easily identified by the dye tracer; however, the bore would collapse when exposed, therefore no information on the shape of the stable bore could be gathered.

Inadvertent drilling fluid returns were only observed on the landside of the levee during the pilot drilling of Bores 1 and 2. The returns were very close to the exit point of the drilling, within 12.2 m (40 ft). The autopsy revealed that the returns were transported through vertical cracks that followed the pipe alignment. However, upon exposing the entire vertical crack, it was noted that the vertical returns were confined by the water table. This is likely due to the fact that the drilling took place in noncohesive soils. Above the water table, the nonsaturated soil exhibited tensile strength, allowing a crack to propagate. However, in the saturated state, the soil would not exhibit any tensile strength and a crack would not form. Therefore, the hydrofracture was confined by the water table.

Hydrofracture was observed at only one other location, within 6 m (20 ft) of the exit point of Bore 1. At this location a horizontal crack, filled with drilling mud, was observed. The crack manifested as a pink seam of drilling mud less than 0.6 cm (1/4 in.) thick. The radial extent of the crack was 2.0 m (6.5 ft). At the location of the crack, it was noted that a thin seam of clay approximately 7.6 cm (3 in.) thick existed in the sandy substratum. The drilling mud infiltrated at the clay/sand interface. This hydrofracture was deemed a very minor event due to the small volume of drilling fluid and confined area of less than 2.1 m (7 ft).

On the riverside of the levee, no inadvertent returns were observed on the surface (or in the subsurface) as continuous mud flow to the pit at the drill rig was maintained throughout the drilling process. The autopsy revealed no sign of hydrofracture, vertically or horizontally, on any of the bores.

9 Conclusions

Prior to the field trials, the conceptual study was completed. The primary conclusions of the conceptual study and preliminary analyses were:

a. Hydrofracture occurs before plastic yielding at shallow depth.

b. Plastic yielding occurs at great depth.

c. As k_o, the coefficient of earth pressure at rest, approaches unity, the stresses required to cause hydrofracture and plastic yielding are equal.

d. Hydrofracture has not been observed in some cases where the theoretical pressure required to cause hydrofracture has been exceeded.

The field tests evaluating the effects of HDD on surrounding soil and ground stability were successfully completed. The data collected during the construction, site investigation, and exhaustive postconstruction autopsy provided detailed information about the interaction between soil characteristics, machine-drilling fluid behavior, and the influences exerted on the levee and subsurface geologic setting. Some of the most intriguing observations are summarized in the following paragraphs.

The drilling fluid pressures inside the drill stem ranged from 1,586 to 2,344 kN/m^2 (230 to 340 psi) on Pilot Bore 1, from 689 to 1,724 kN/m^2 (100 to 250 psi) on Pilot Bore 2, and from 1,206 to 2,068 kN/m^2 (175 to 300 psi) on Pilot Bore 3. The variations in internal pressures were planned to allow evaluation of the effects of drilling fluid pressures on the levee and subsurface soil mass. One striking result was that, regardless of the internal nozzle pressure, the external pressures, measured 0.3 m (1 ft) behind the nozzle in the annulus, were within a very narrow range (324 to 358 kN/m^2 (47 to 52 psi)) for all three pilot bores. The external pressures were 17 to 21 percent of the internal pressures.

The internal drilling pressures measured on the reaming bores were much lower than those for the pilot bores, (approximately one-half the magnitude). However, the external pressures remained within the same narrow band of 324 to 358 kN/m^2 (47 to 52 psi) for both reaming operations.

The piezometric pressure increases observed during the pilot boring operations were typically 3.4 kN/m^2 (0.5 psi) or less. These pressure increases correspond to

less than 1.0 percent of the external annular pressures. The increases in piezometric pressures were very transient, lasting less than ½ hr while the drilling head was in close proximity to the piezometer. Piezometric pressure fluctuations associated with changes in river stages before construction greatly exceeded those associated with HDD construction. Piezometric pressure increases associated with HDD construction should not be of concern for levee stability.

Further, the results of the experiment were observed at a location where ground conditions and geometry would be considered relatively unfavorable, compared with conditions normally sought for levee or river crossings. Typically, owners, permitting agencies, designers, and contractors prefer deep crossings with substantial thicknesses of clay between the bore path and the levee or river bottom. Substantial clay cover is believed to minimize the risk of inadvertent fluid returns and other potential problems. This experiment clearly established the degree of conservatism usually inherent in design of HDD crossings. On this project, maximum depth of cover was approximately 13.5 m (45 ft) at the levee center line, with typical depths at the riverside and landside toe of 7.5 m (25 ft). With a substratum composed of primarily silty sand, these conditions would not be considered ideal site conditions for an HDD crossing. Yet, the impact as measured by the piezometers was minimal.

Postconstruction excavation revealed boreholes that were stable and lined with a bentonite mud cake above the water table. Below the water table, the boreholes were stable and filled with drilling fluid. Over the vast majority of the drilling, the drilling fluid remained within the borehole and did not radiate away from the bore path.

During the autopsy portion of the project, very few signs of hydrofracture were observed. The only significant area of hydrofracture was observed on Bore 2 on the landside of the levee, within 15 m (50 ft) of the exit point. At this location, hydrofracture manifested as a vertical crack, above the pipeline, that extended to the ground surface. However, the hydrofracture did not occur until the drilling head exited the water table, within 3.3 m (11 ft) of the ground surface. Inadvertent returns were observed at this location, 6.0 m (20 ft) from the exit of the bore. By all observations, the water table appeared to have confined the hydrofracture.

At all other excavation locations, the only zones of hydrofracture occurred within 3.0 m (10 ft) of the exit or entry location. Throughout the full range of drilling pressures used on the project, hydrofracture was not a significant problem. These visual results further substantiate and generally support the conclusions obtained from the piezometric data.

10 Recommendations

The recommended guidelines for the installation of pipelines with HDD are contained in Appendix A of this report. The recommendations are based on the results and conclusions of the Construction Productivity Advancement Research Program field evaluation, as well as analytical studies of soil/drilling fluid interaction and evaluations of case histories. The recommendations address the main issues of concern that have been expressed by USAED personnel, either in USAED regulations or in meetings and discussions.

11 Commercialization and Technology Transfer

The most significant product of this research is the findings contained within this report and the Recommended Guidelines for Installation (Appendix A), based on the results of the field studies, in combination with case history evaluations and studies of soil-drilling fluid interaction. This report will be utilized by decision makers to make informed decisions on the use of HDD to provide for a safe and secure pipeline levee crossing. Key elements addressed in the recommended guidelines include:

a. Establishing allowable drilling fluid pressures.

b. Pressure monitoring techniques.

c. Appropriate setback distances.

d. Establishing appropriate depths of cover over the pipeline.

e. Speed of drilling.

f. Effects of groundwater.

g. Prevention of seepage and erosion.

h. Closure devices.

i. Use of relief wells.

To expedite the technology transfer process, numerous preliminary reports were prepared and presented throughout the life of the project. These technical papers and presentations included the following:

North American Society of Trenchless Technology, No-Dig '97, Seattle, WA
Corps of Engineers Geotechnical Conference, San Bernadino, CA
ASCE, Georgia Section Meeting, Atlanta, GA
ASCE Construction Congress, Minneapolis, MN
Worldwide Area Engineers Conference, Orlando, FL
Underground Construction Technologies Conference, Houston, TX

The final report will be reproduced and distributed to the pipeline industry by the American Gas Association's Pipeline Research Committee International (AGA-PRC). All member companies of the AGA will receive copies of the final report and guidelines for installation. Additional copies will be made available upon request by the Corps of Engineers and the American Gas Association.

References

American Gas Association, Pipeline Research Committee. (1994). "Drilling fluids in pipeline installation by horizontal directional drilling." *Practical application manual.* PR-227-9321, Tulsa, OK.

Delft Geotechnics. (1997). A report by Department of Foundations and Underground Engineering prepared for O'Donnell Associates, Inc., Sugarland, TX.

Hair, J. D., and Associates, Inc., Cappozoli, Louis J., and Associates, Inc., and Stress Engineering Services, Inc. (1995). "Installation of pipelines by horizontal directional drilling." *Engineering design guide.* PR-227-9424, Tulsa, OK.

Headquarters, U.S. Army Corps of Engineers. (1965). "Interior drainage of leveed urban areas: Hydrology," Engineer Manual 1110-2-1410, Washington, DC.

__________. (1978). "Design and construction of levees," Engineer Manual 1110-2-1913, Washington, DC.

Jaworski, G. W., Duncan, J. M., and Seed, H. B. (1981). "Laboratory study of hydraulic fracturing," *Journal of the Geotechnical Engineering Division, Proceedings of the American Society of Civil Engineers* 107(GT6).

Luger, H. J., and Hergarden, H. J. A. M. (1988). "Directional drilling in soft soils: Influence of mud pressures." *Proceedings, International No-Dig '88.* Washington, DC.

Pipeline and Gas Journal. (1996). "Directionally drilled crossing constructed under river levee," 223(6), 26-28.

Sauls, David P., and Hair III, Charles W. (1996). "Geotechnical support embankment subdrilling study, Mississippi River Mile 492.9 AHP, Issaquena County, Mississippi," Louis J. Cappozoli & Associates, Inc., Baton Rouge, LA.

Third International Conference and Exhibition on Trenchless Technology. (1988). "No-Dig '88: Achievements in trenchless technology," Conference Proceedings, Washington, DC.

U.S. Army Engineer District, Vicksburg. (1993). "Project operations - Project maintenance standards and procedures," District Regulation 1130-2-303, Vicksburg, MS.

Wells, J. T., and Kemp, G. P. (1981). "Atchafalaya mudstream and recent mudclat progradation; Louisiana chenier plain," *31st annual meeting, Gulf Coast Association of Geological Societies and 25th annual meeting of Gulf Coast Section of Society of Economic Paleontologists and Mineralogists*. Corpus Christi, TX.

Yanagisawa and Komak Panah. (1994). "Two-dimensional study of hydraulic fracturing criteria in cohesive soils," *Soils and Foundations Journal* 34(1), 1-9.

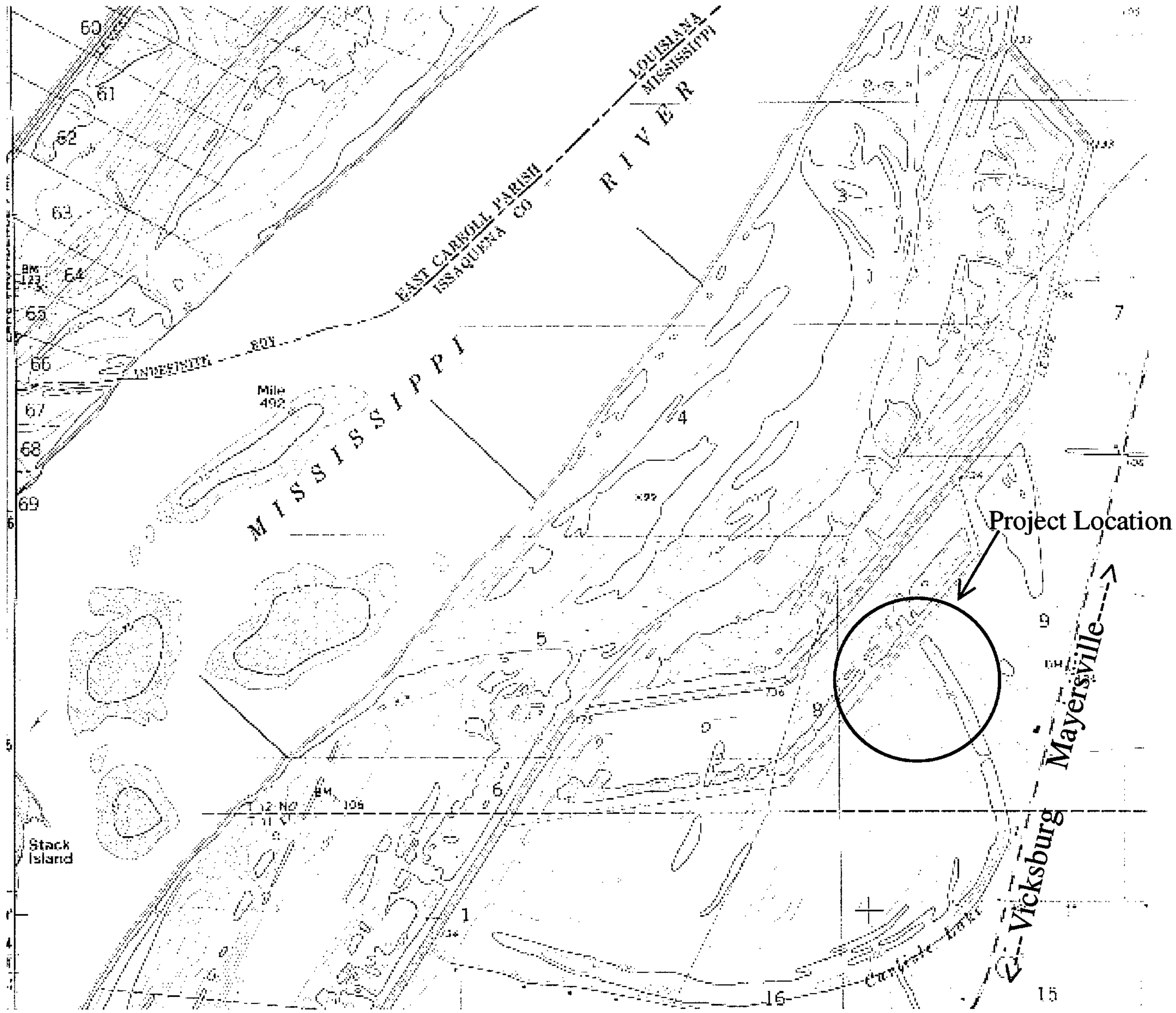


Figure 1. Whiting Bayou quadrangle map

Figure 2. Aerial photo of the set-back levee and test site

Figure 3. Lake Carlisle as seen from setback levee and exit site for test bores

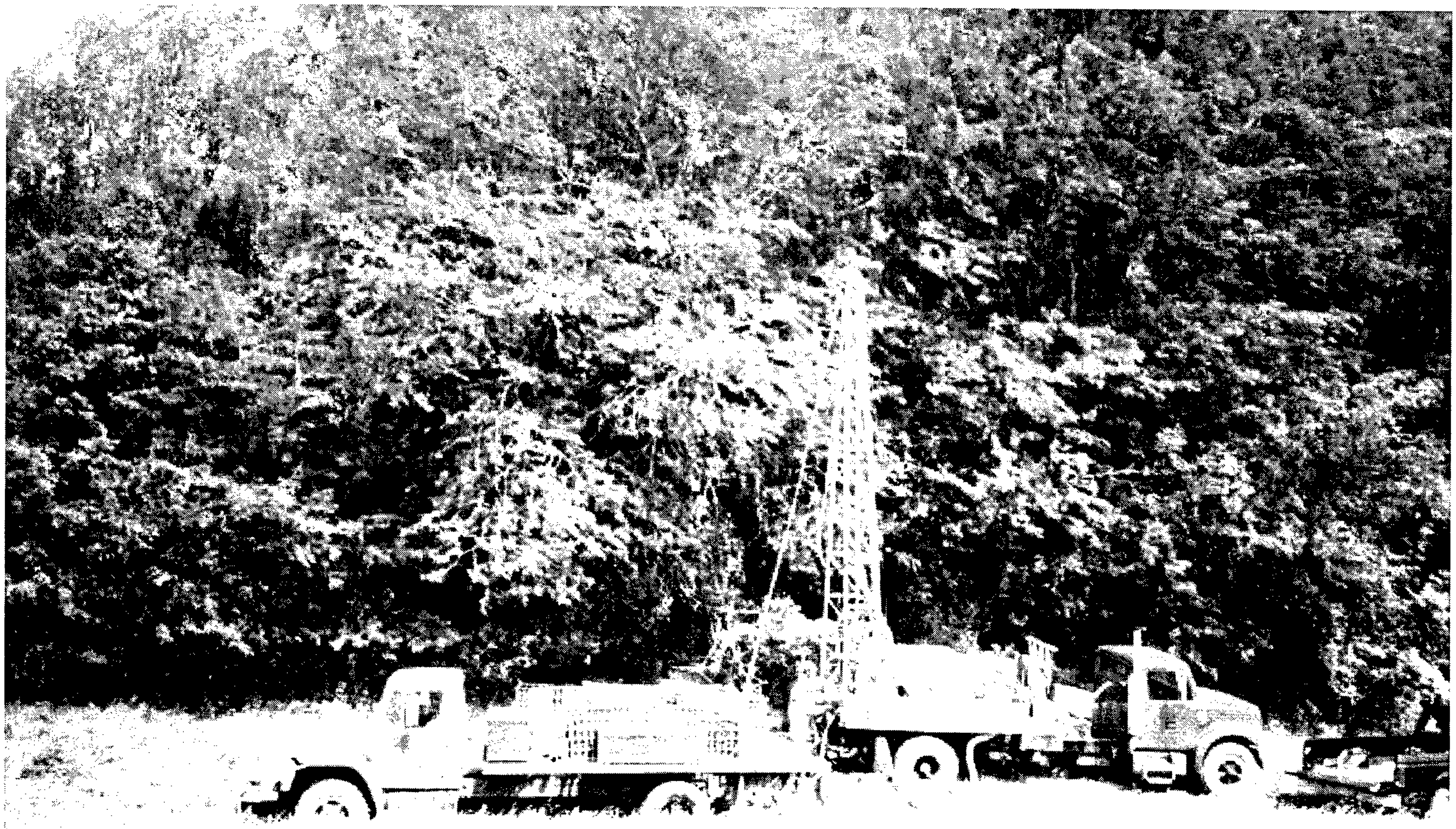


Figure 4. Drill rig taking soil samples and installing piezometers

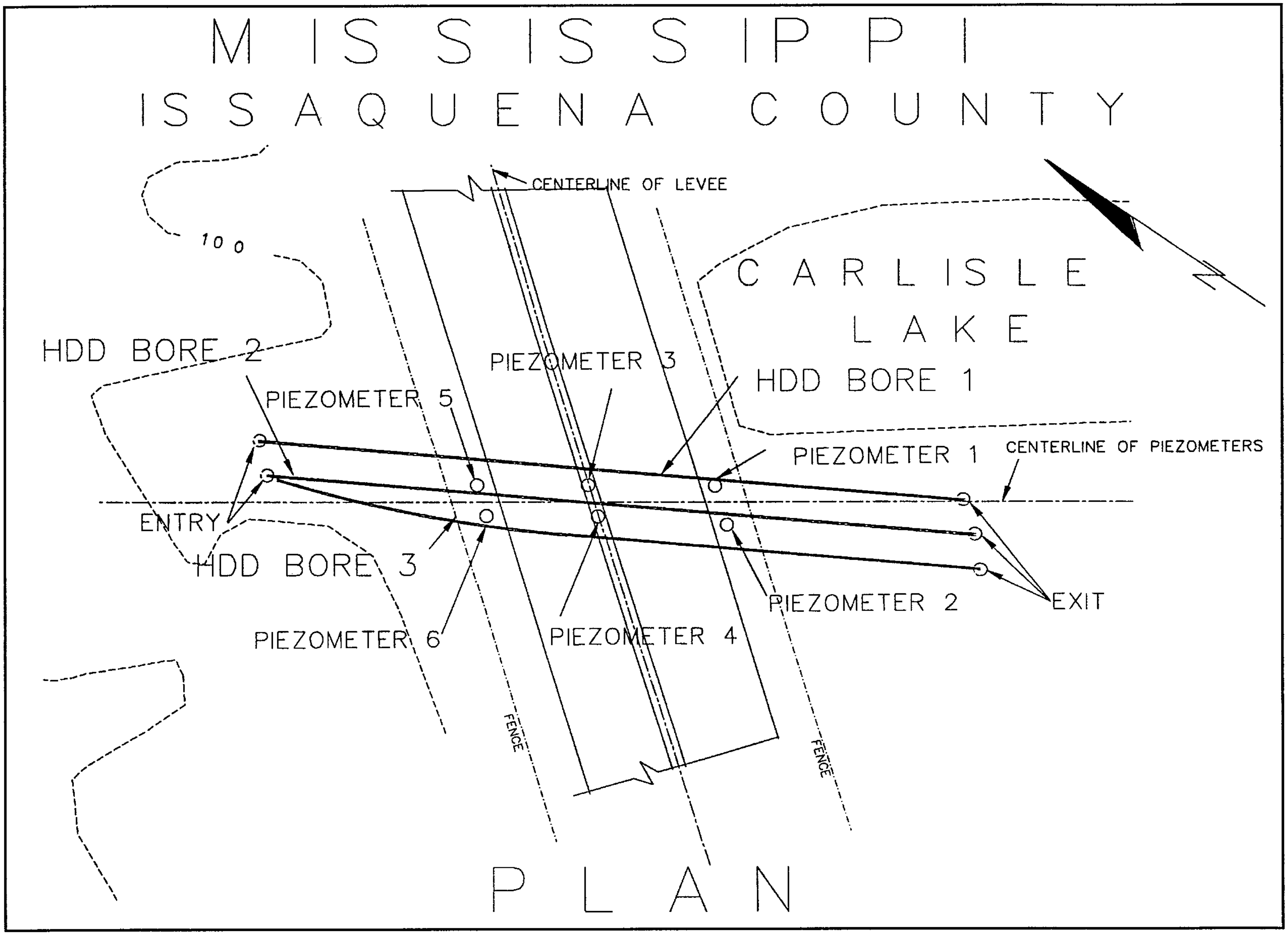
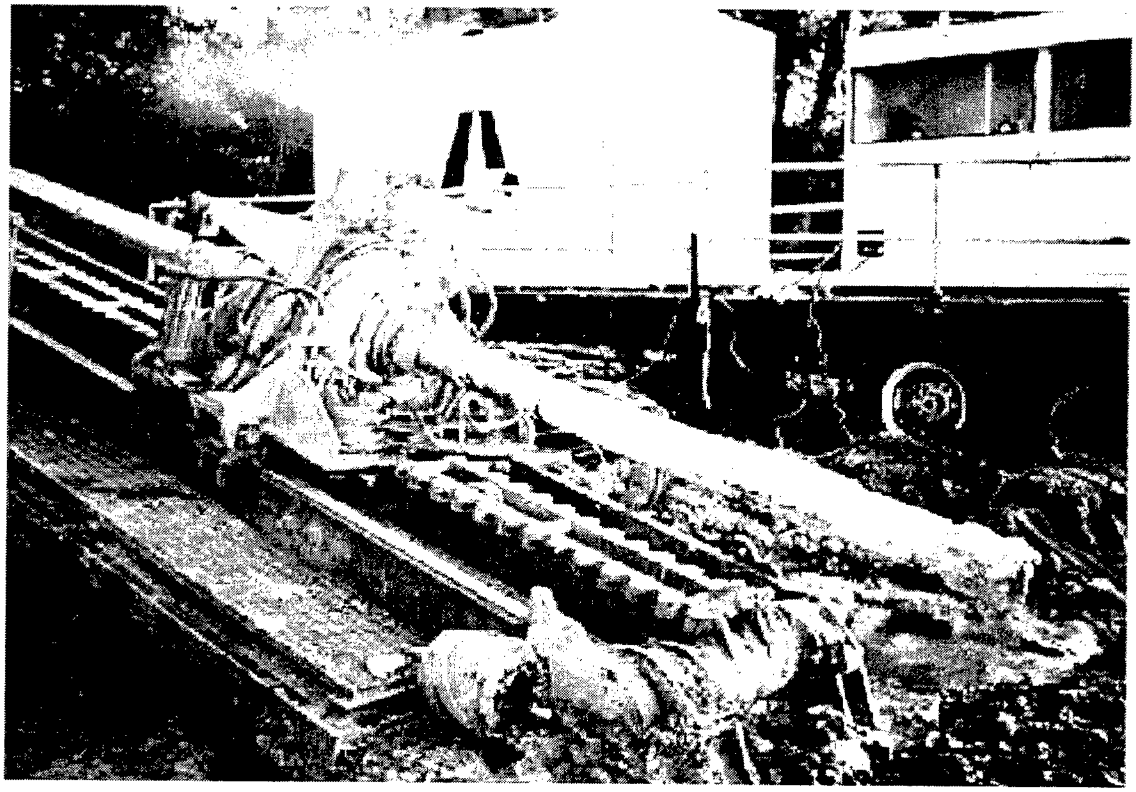


Figure 5. Plan view of test site detailing pipelines and piezometer locations



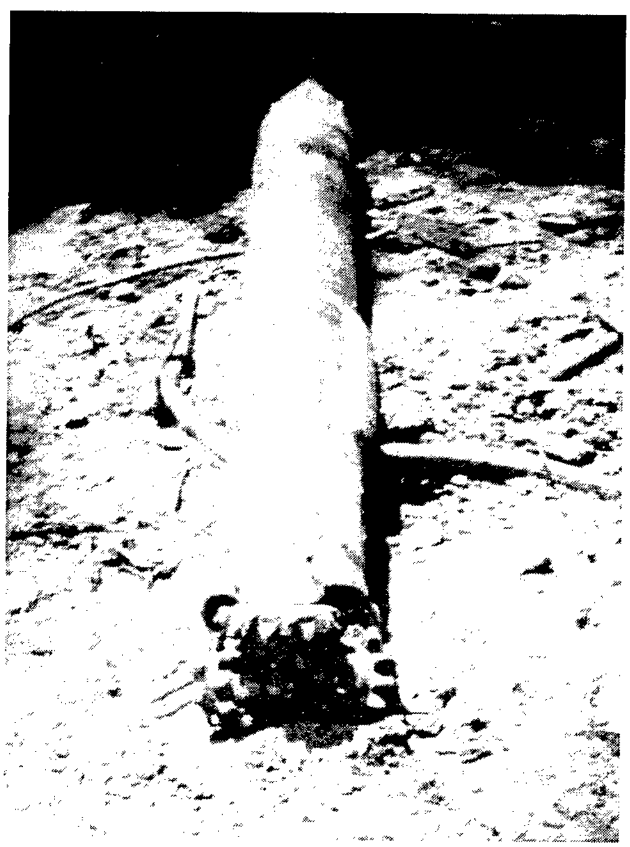


Figure 6. Drill rig (upper) used for all three bores and tri-cone (lower) with jet nozzles

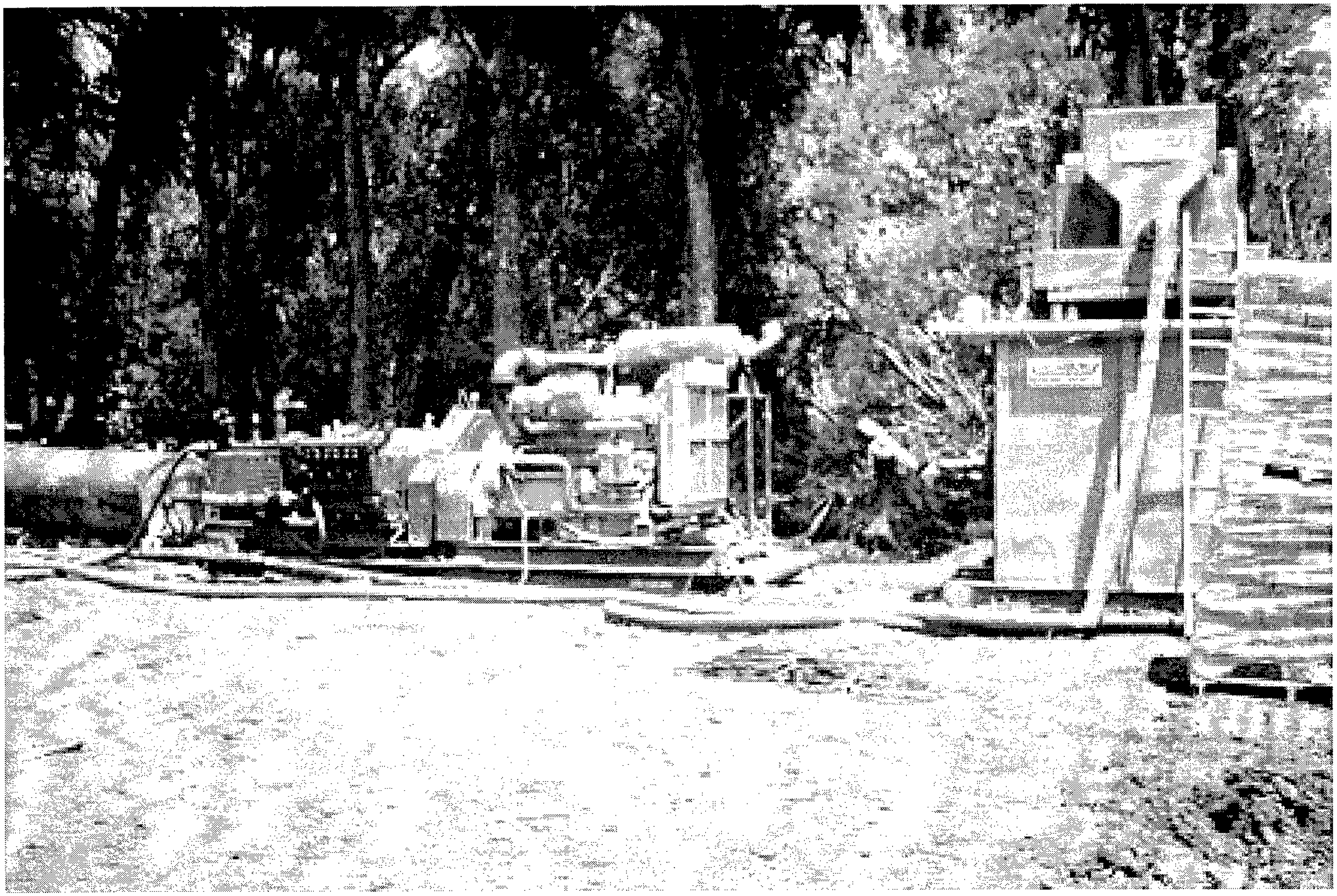


Figure 7. Mud system

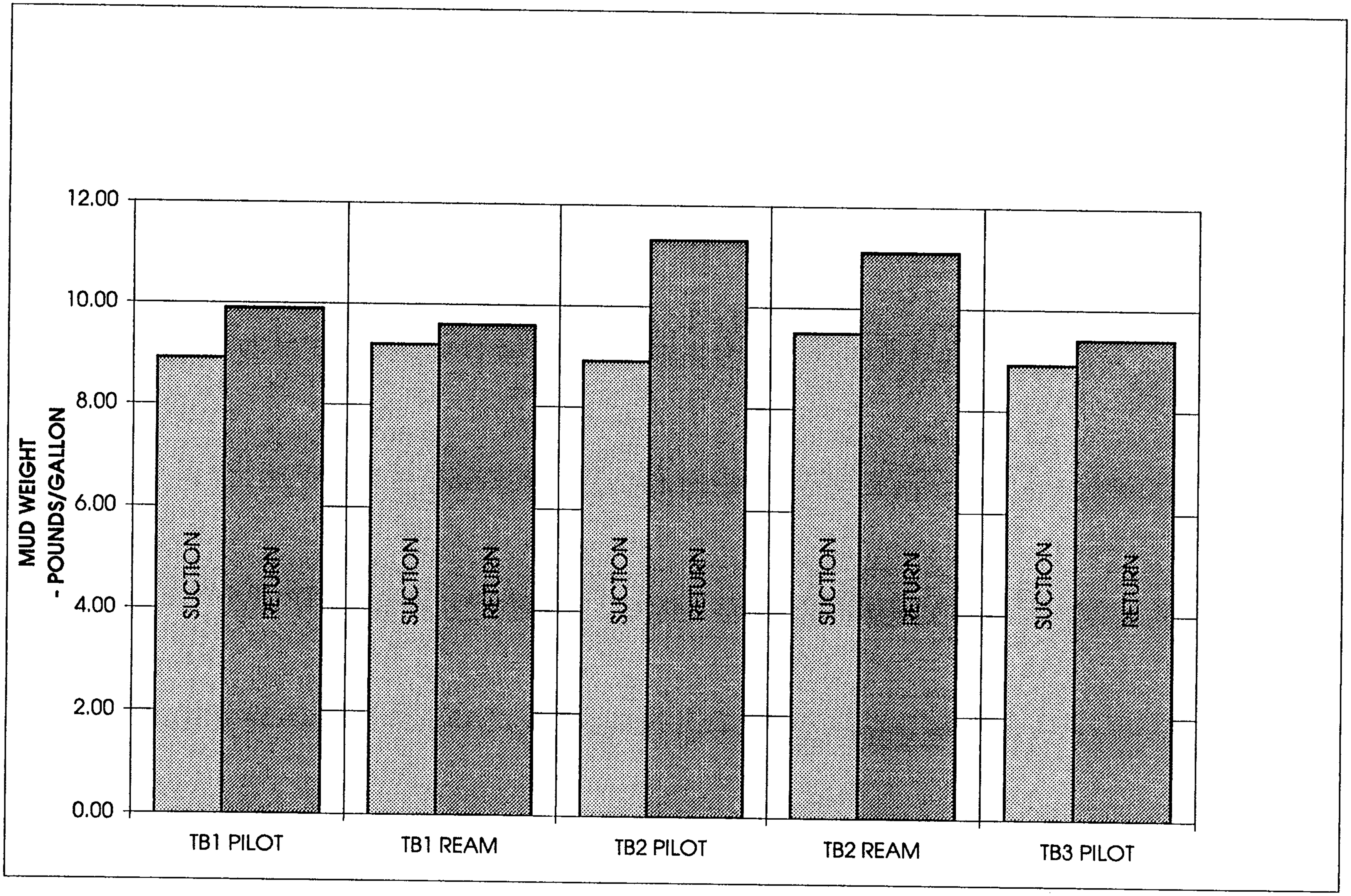


Figure 8. Drilling mud weight - suction and return, Test Bores 1 through 3. (To convert lb/gal to kg/ℓ, multiply by 0.11)

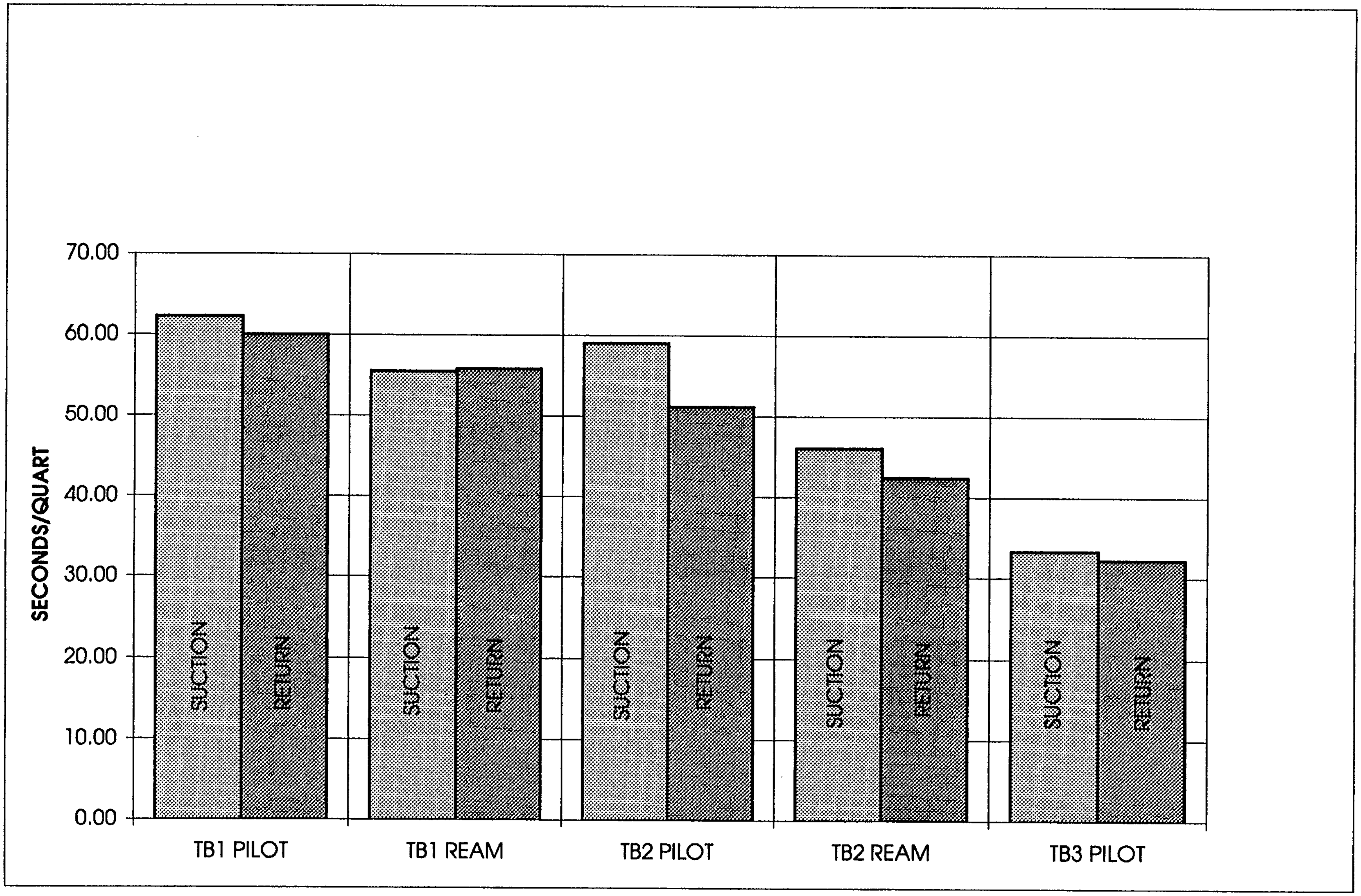


Figure 9. Drilling fluid viscosity - suction and return, Test Bores 1 through 3. (To convert quarts to liters, multiply by 0.95)

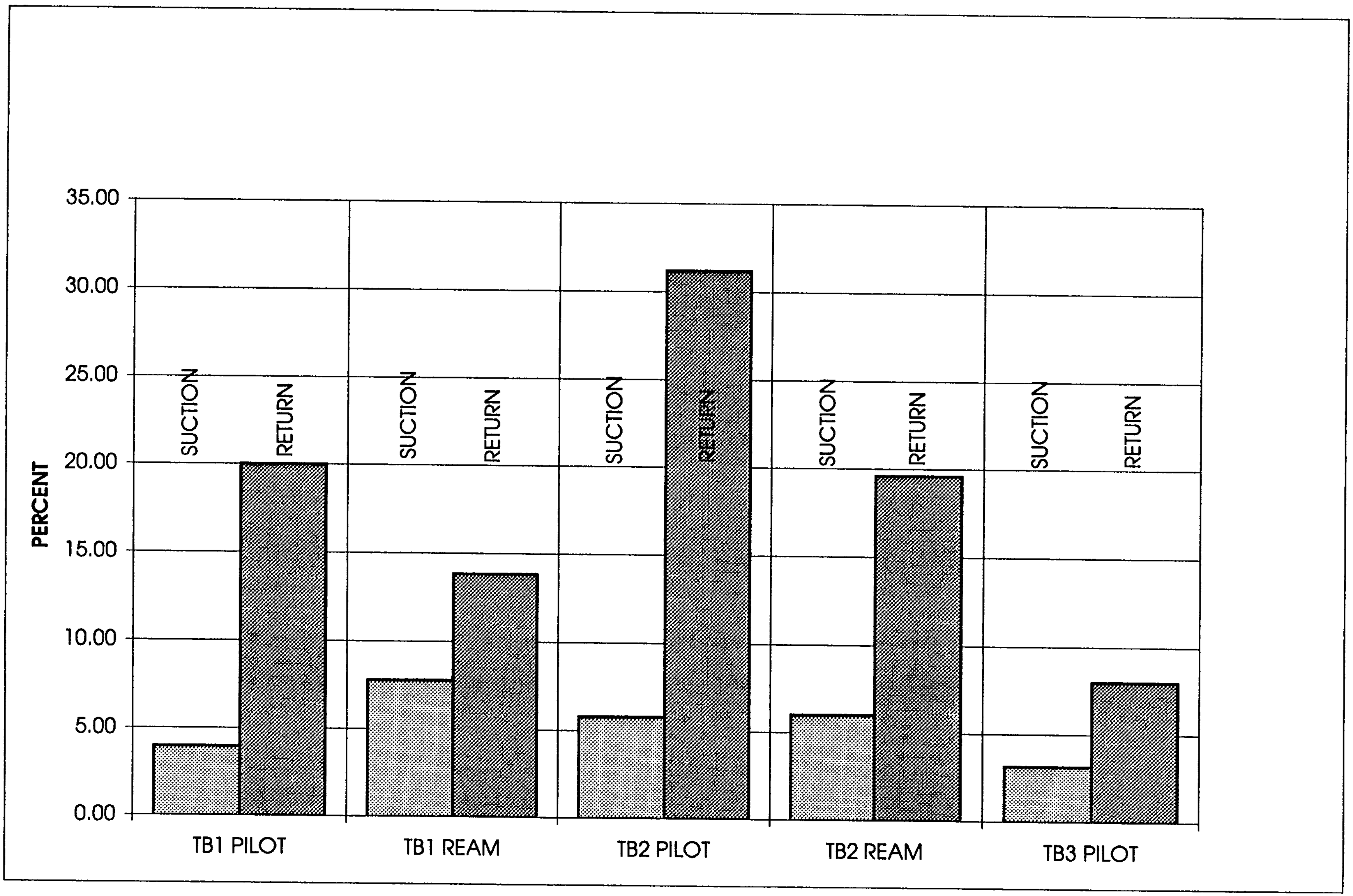


Figure 10. Sand content - suction and return, Test Bores 1 through 3

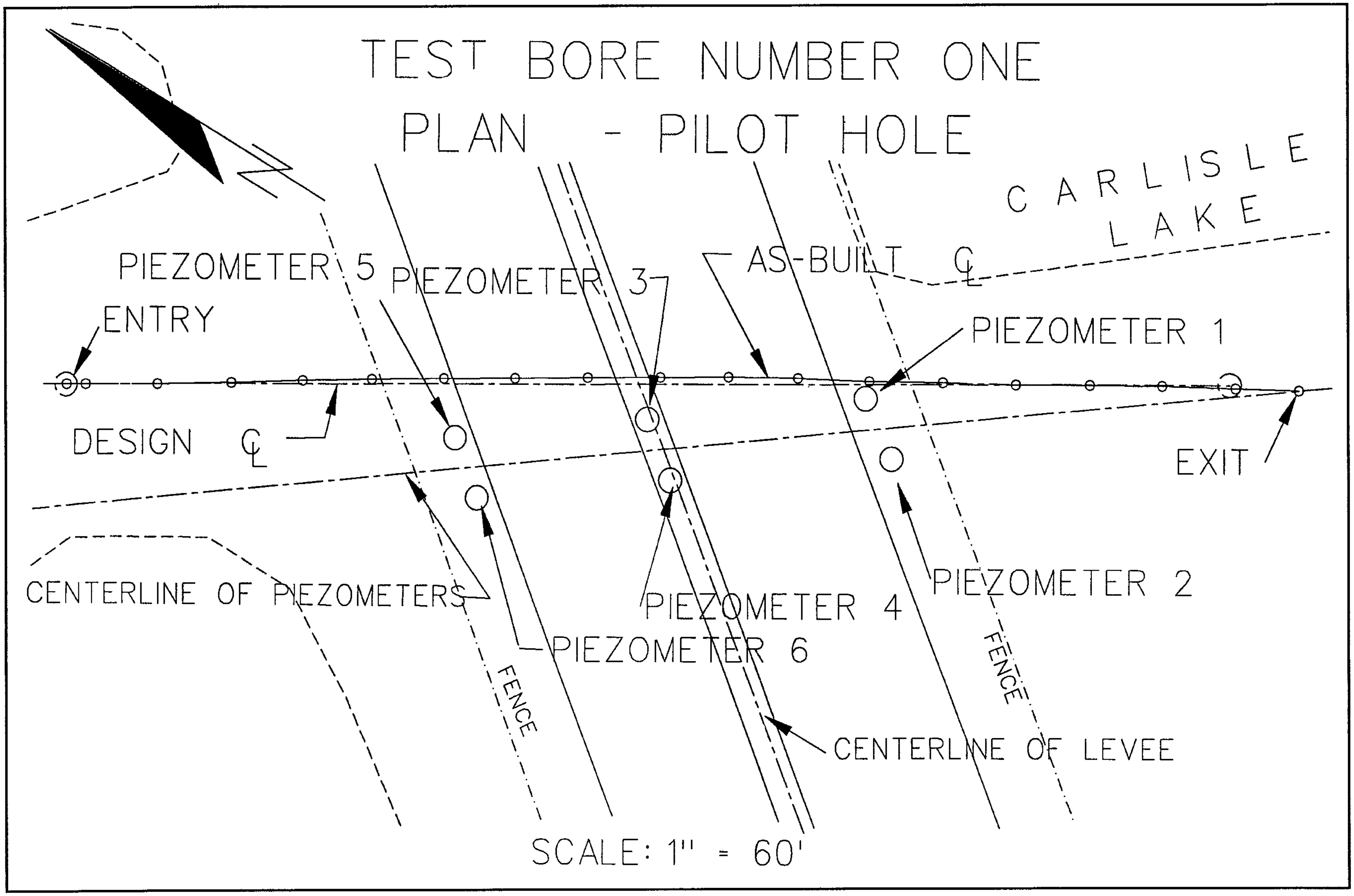


Figure 11. Plan view of Bore 1; planned and as-built location. (To convert inches to centimeters, multiply by 2.54; to convert feet to meters, multiply by 0.305)

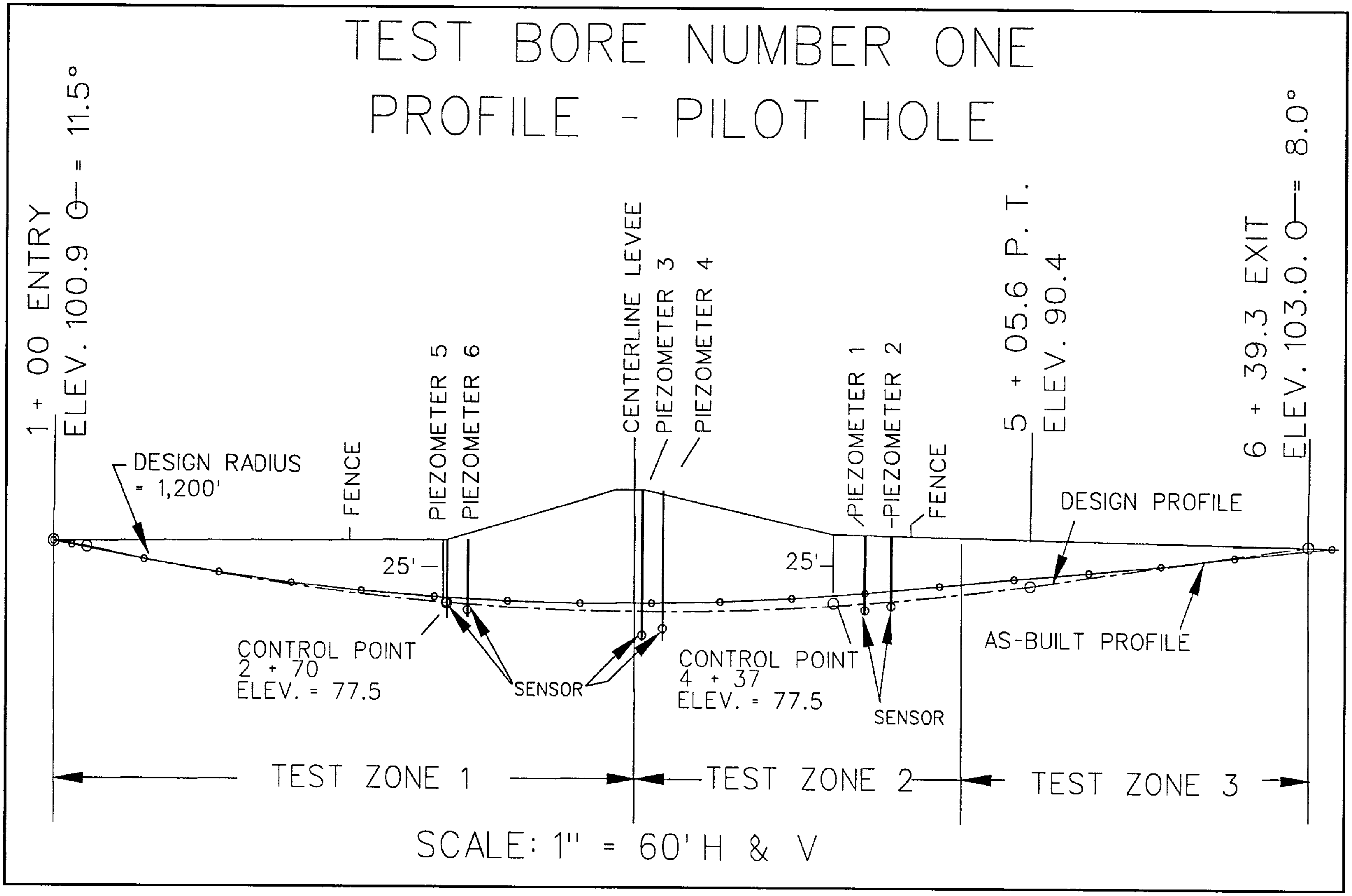


Figure 12. Profile view of Bore 1; planned and as-built location. (To convert inches to centimeters, multiply by 2.54; to convert feet to meters, multiply by 0.305; to convert degrees (angle) to radians, multiply by 0.017)

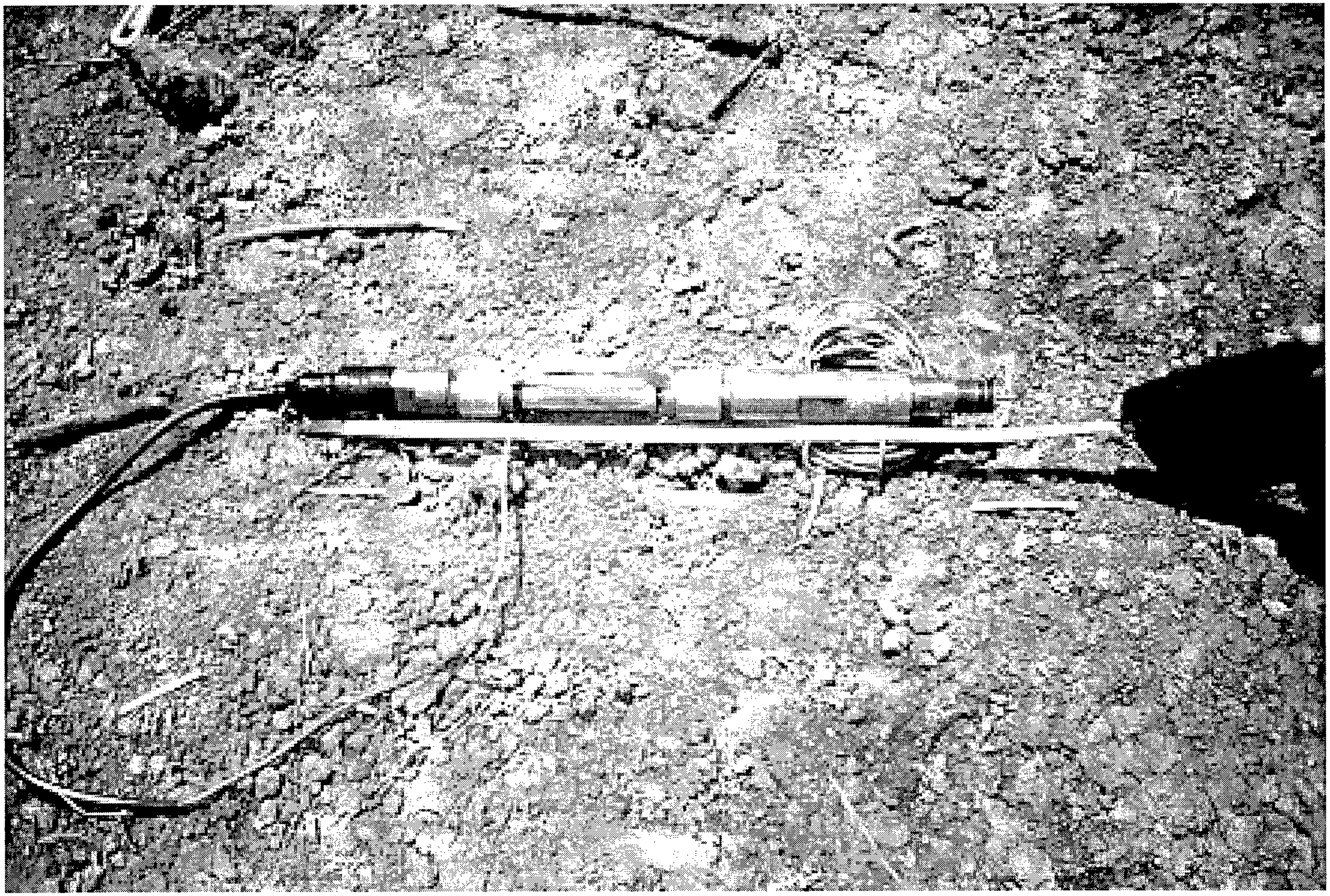


Figure 13. External pressure sensing device

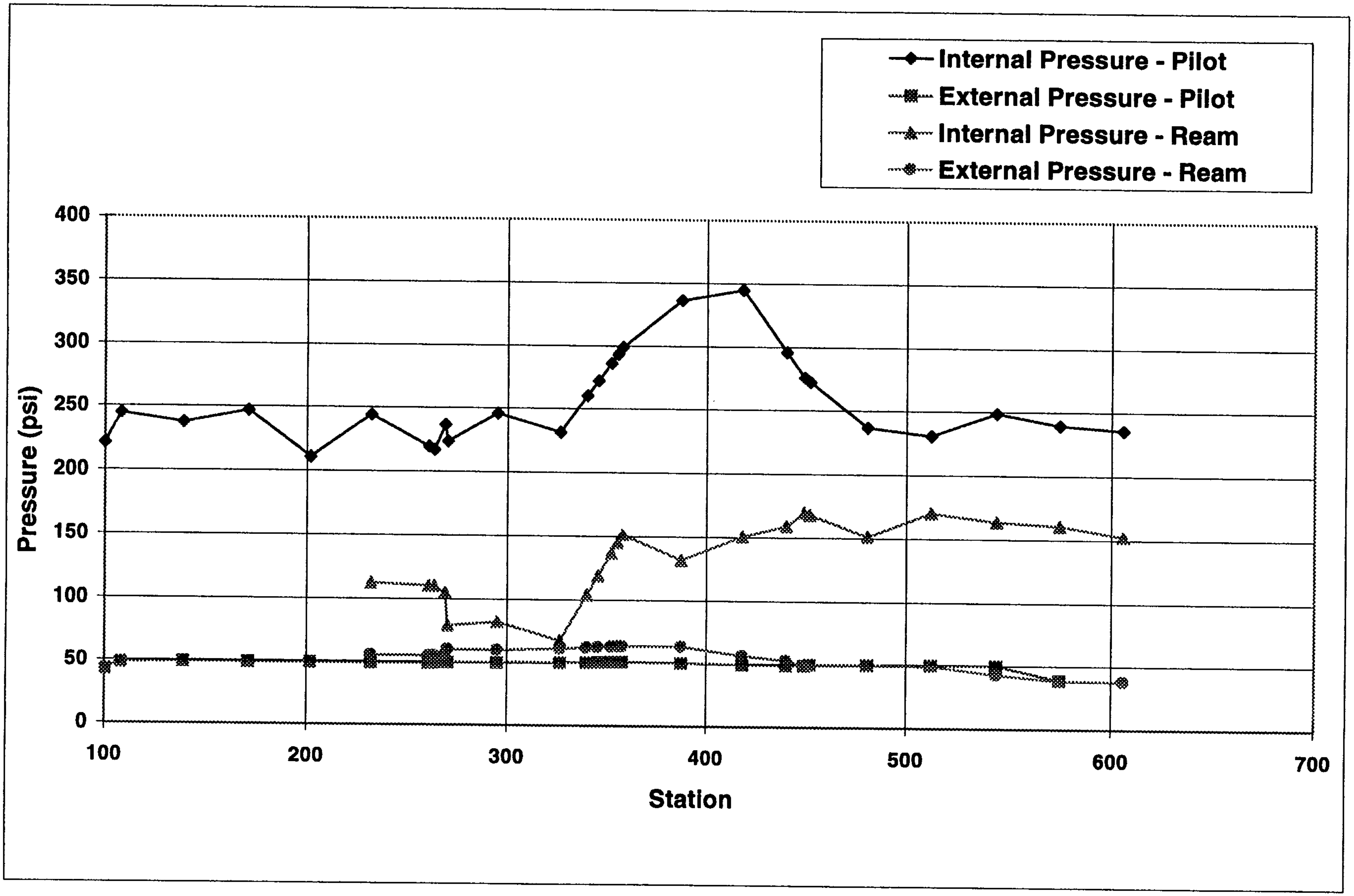


Figure 14. Internal and external pressures measured during pilot drilling and reaming of Bore 1. (To convert psi to kN/m^2, multiply by 6.89)

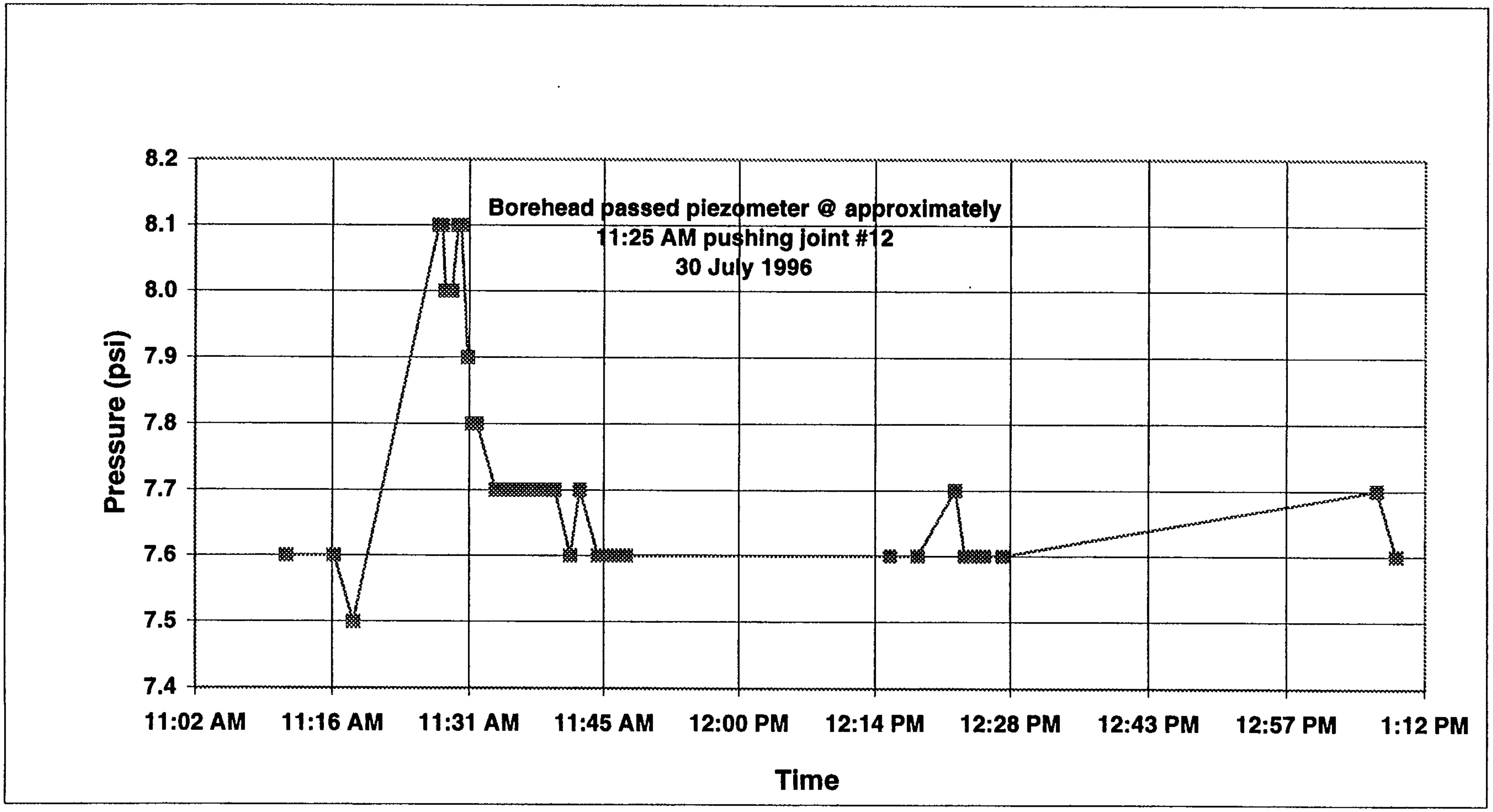


Figure 15. Piezometer 1, Pilot Bore 1. (To convert psi to kN/m^2, multiply by 6.89)

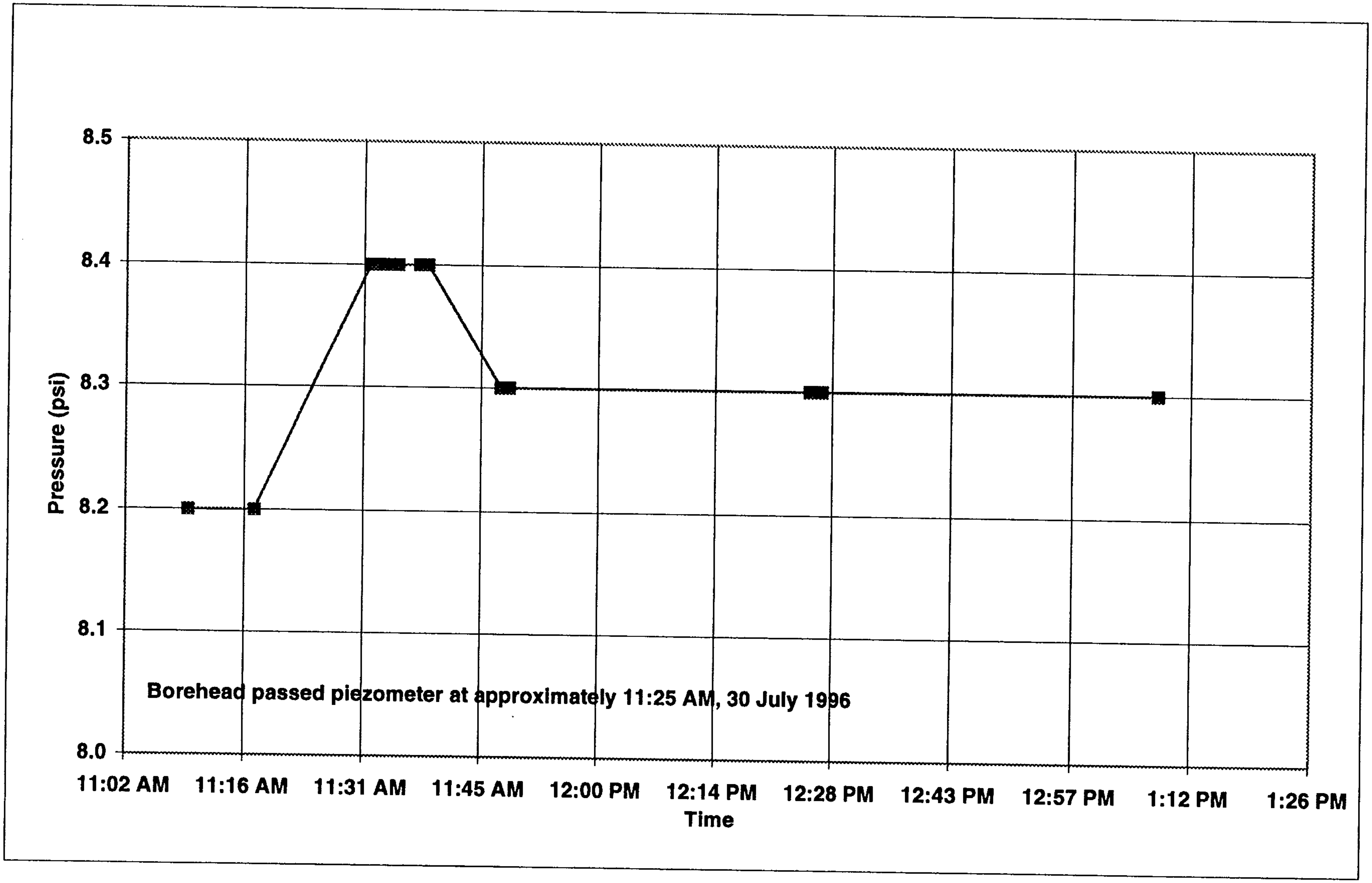


Figure 16. Piezometer 2, Pilot Bore 1. (To convert psi to kN/m², multiply by 6.89)

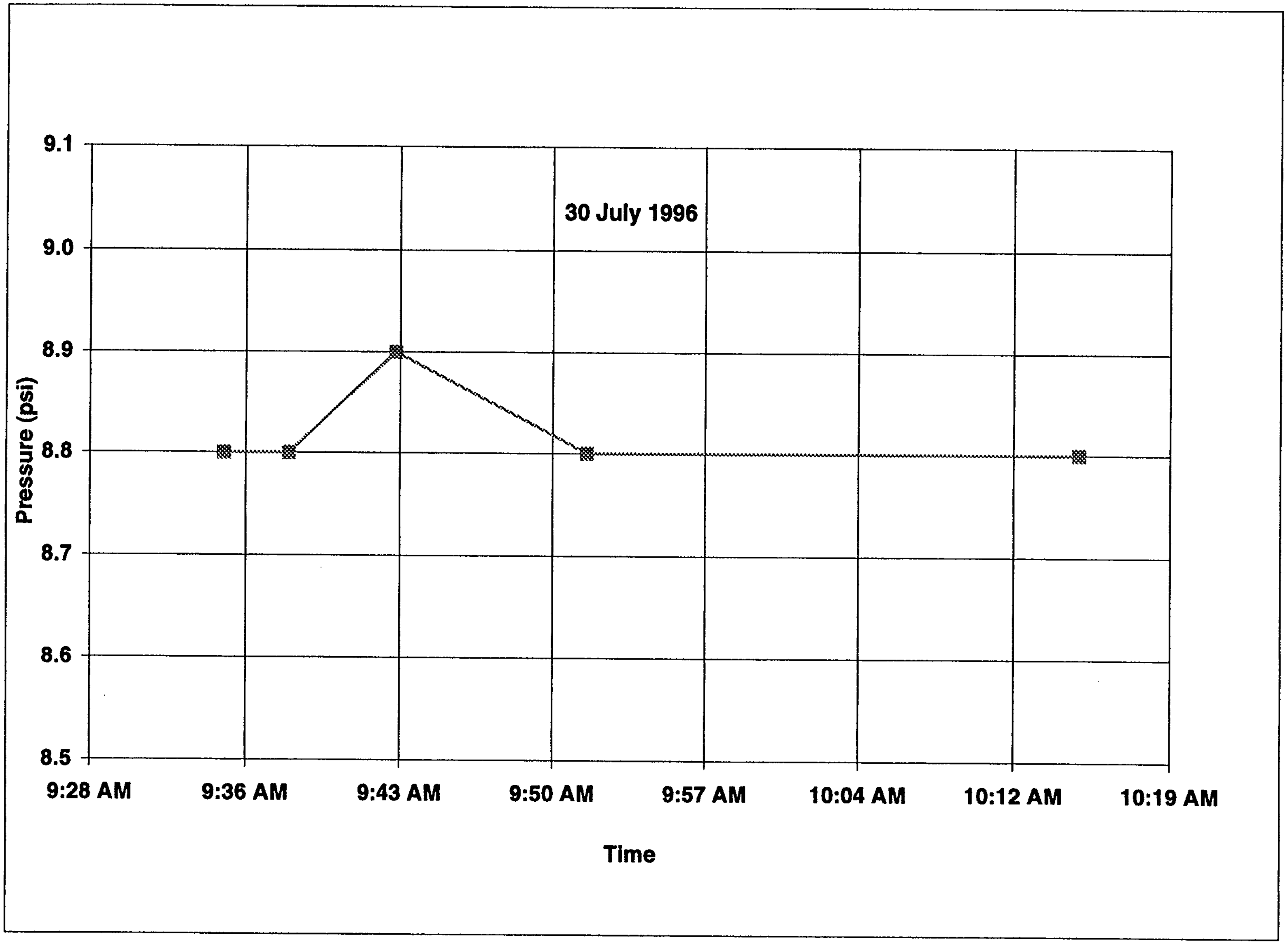


Figure 17. Piezometer 5, Pilot Bore 1. (To convert psi to kN/m^2, multiply by 6.89)

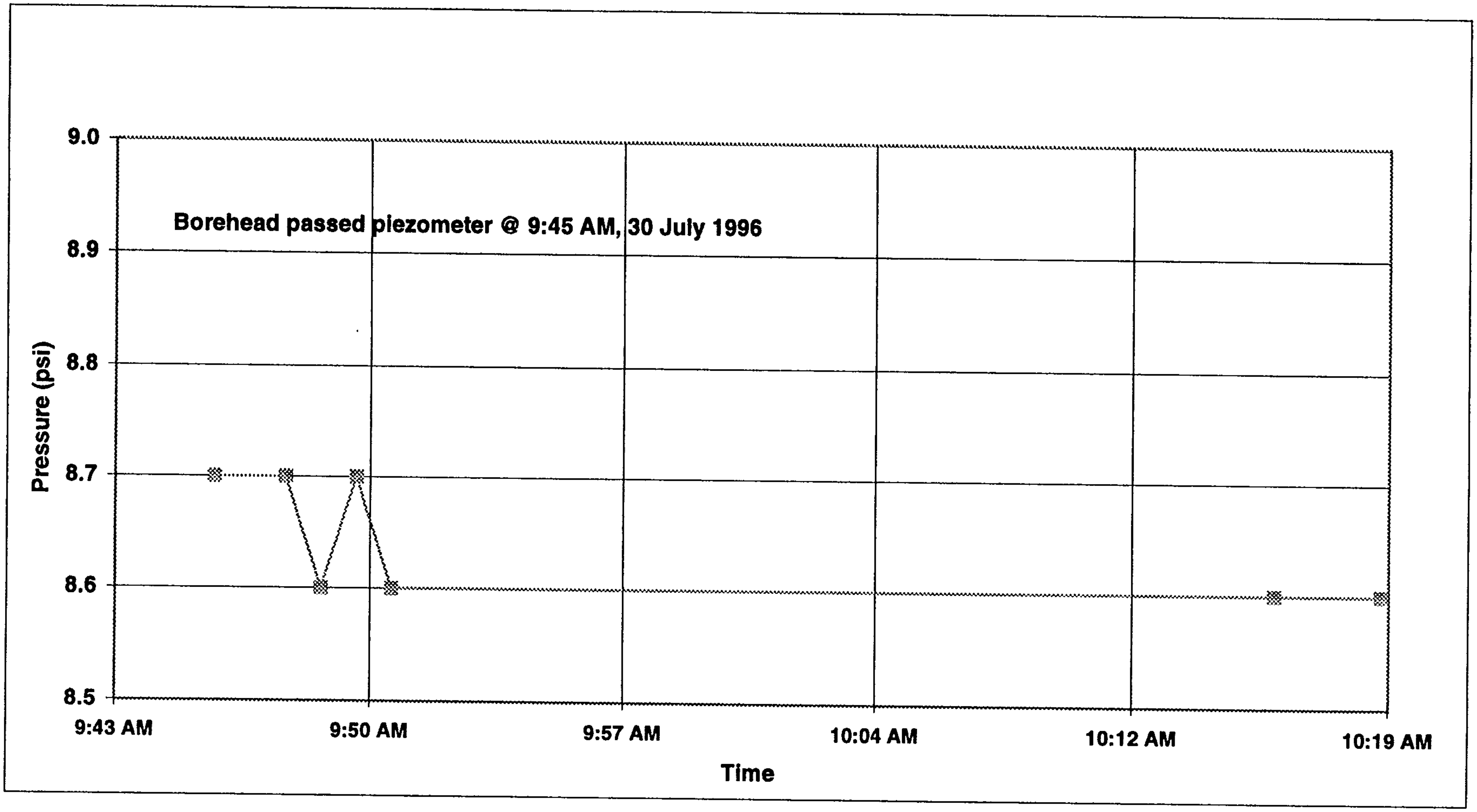


Figure 18. Piezometer 6, Pilot Bore 1. (To convert psi to kN/m^2, multiply by 6.89)

Figure 19. Fly cutter assembly

Figure 20. Fly cutter, drill stem, and pipeline

Figure 21. Installation of the 12-in steel pipe. (To convert inches to centimeters, multiply by 2.54)

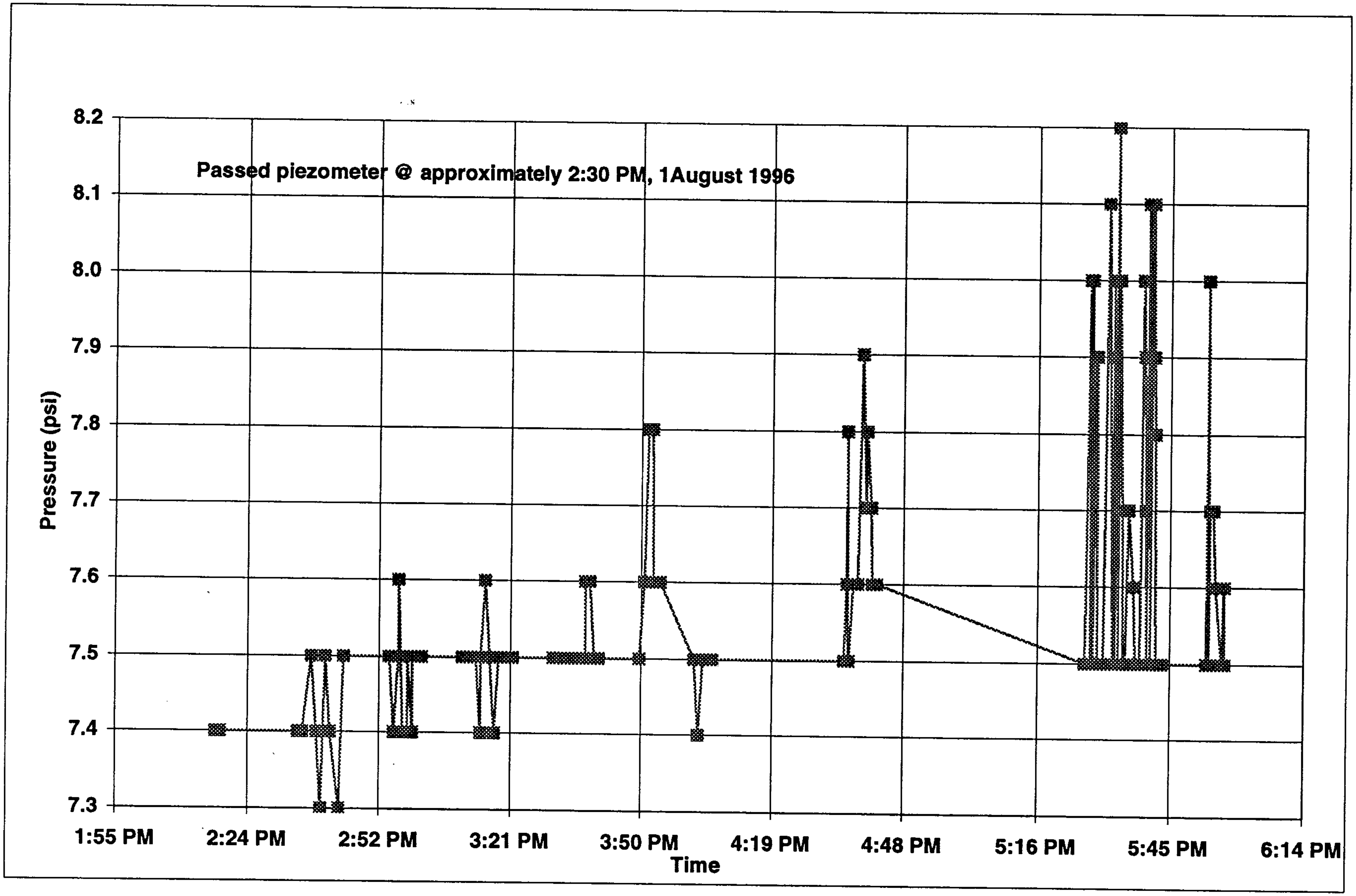


Figure 22. Piezometer 1, Reaming Bore 1. (To convert psi to kN/m^2, multiply by 6.89)

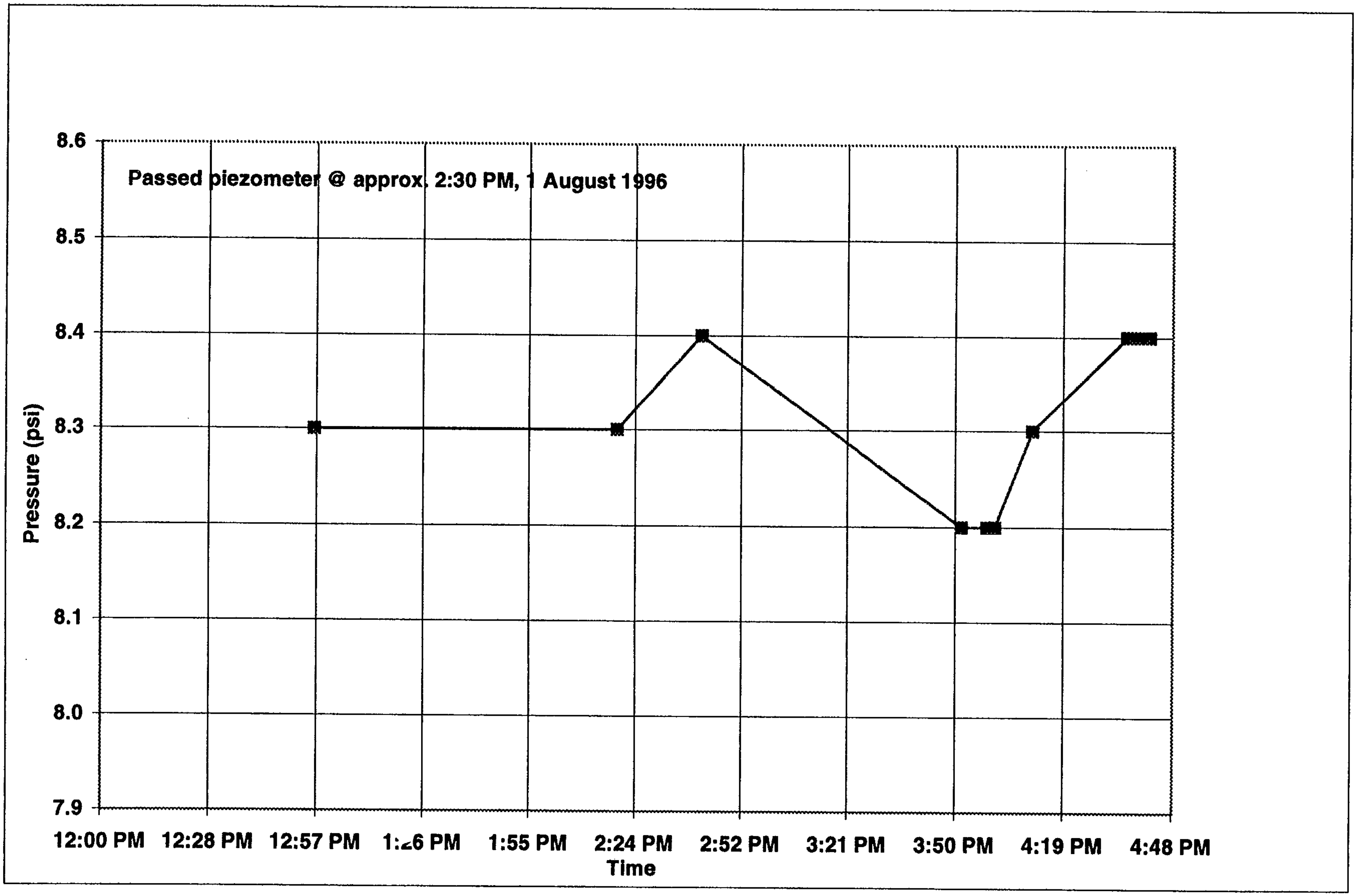


Figure 23. Piezometer 2, Reaming Bore 2. (To convert psi to kN/m², multiply by 6.89)

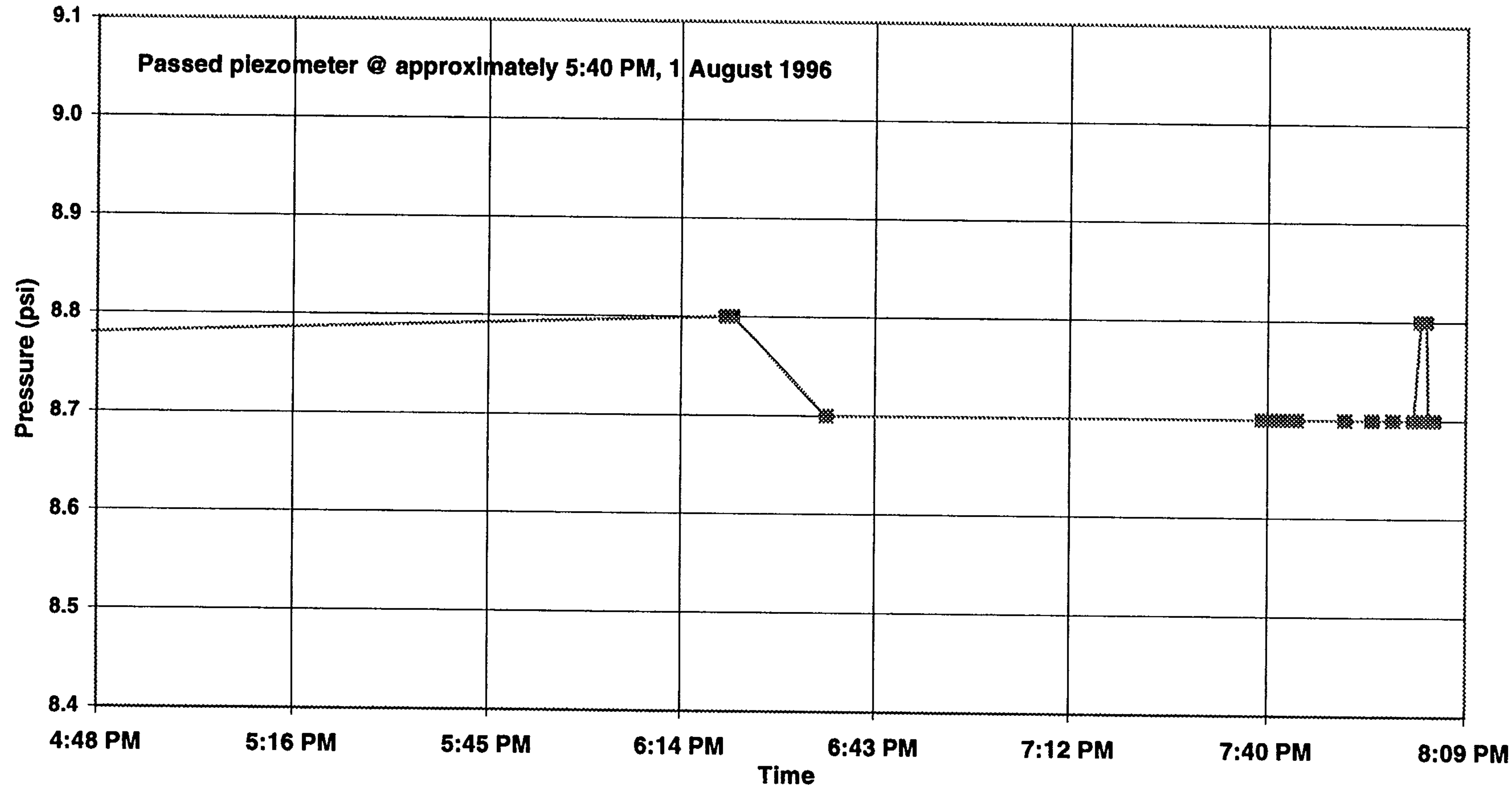


Figure 24. Piezometer 5, Reaming Bore 1. (To convert psi to kN/m^2, multiply by 6.89)

Figure 25. Inadvertent returns on Pilot Bore 2, within 13.2 m (44 ft) of the exit

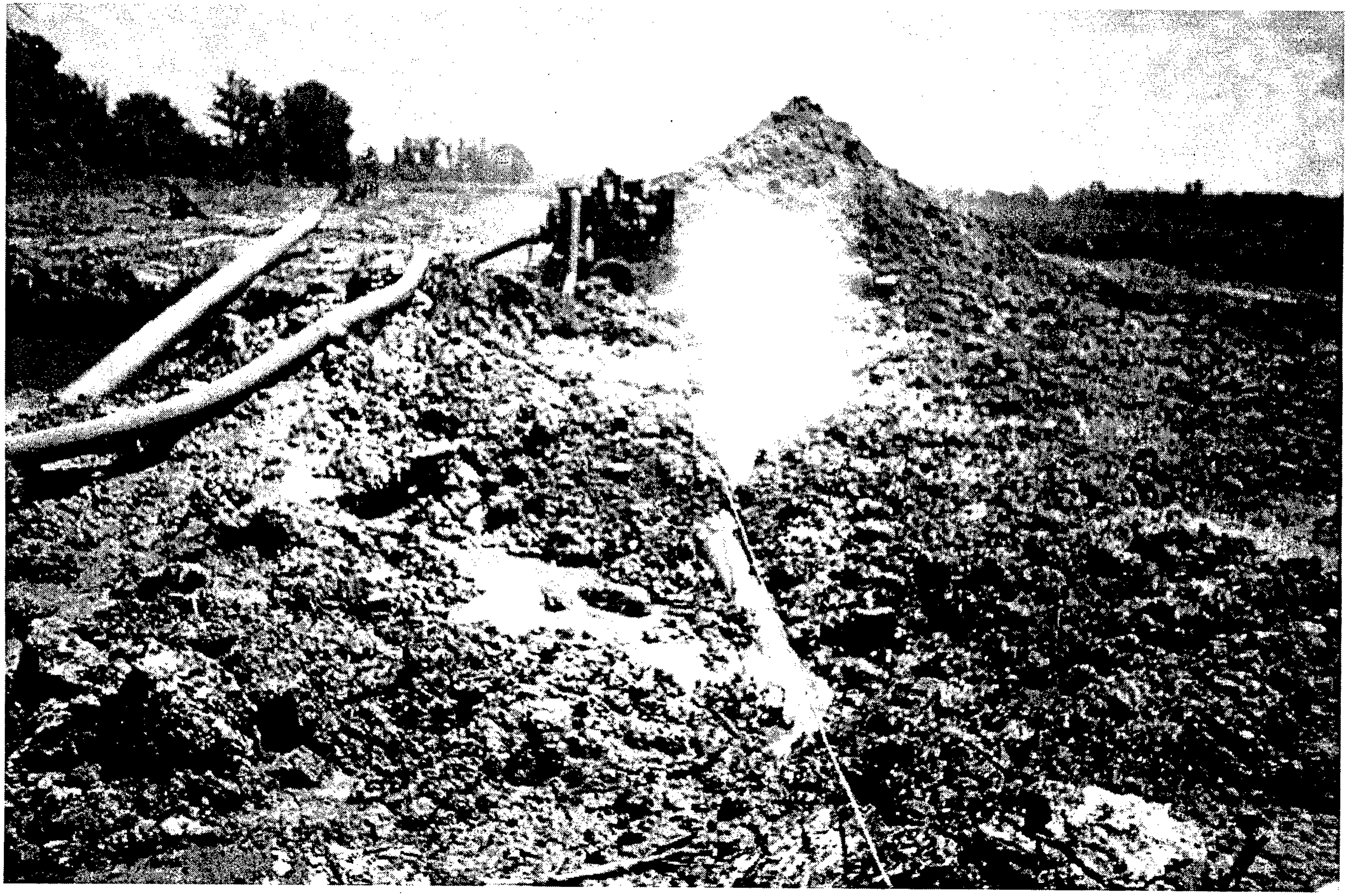


Figure 26. Punch out of Pilot Bore 2

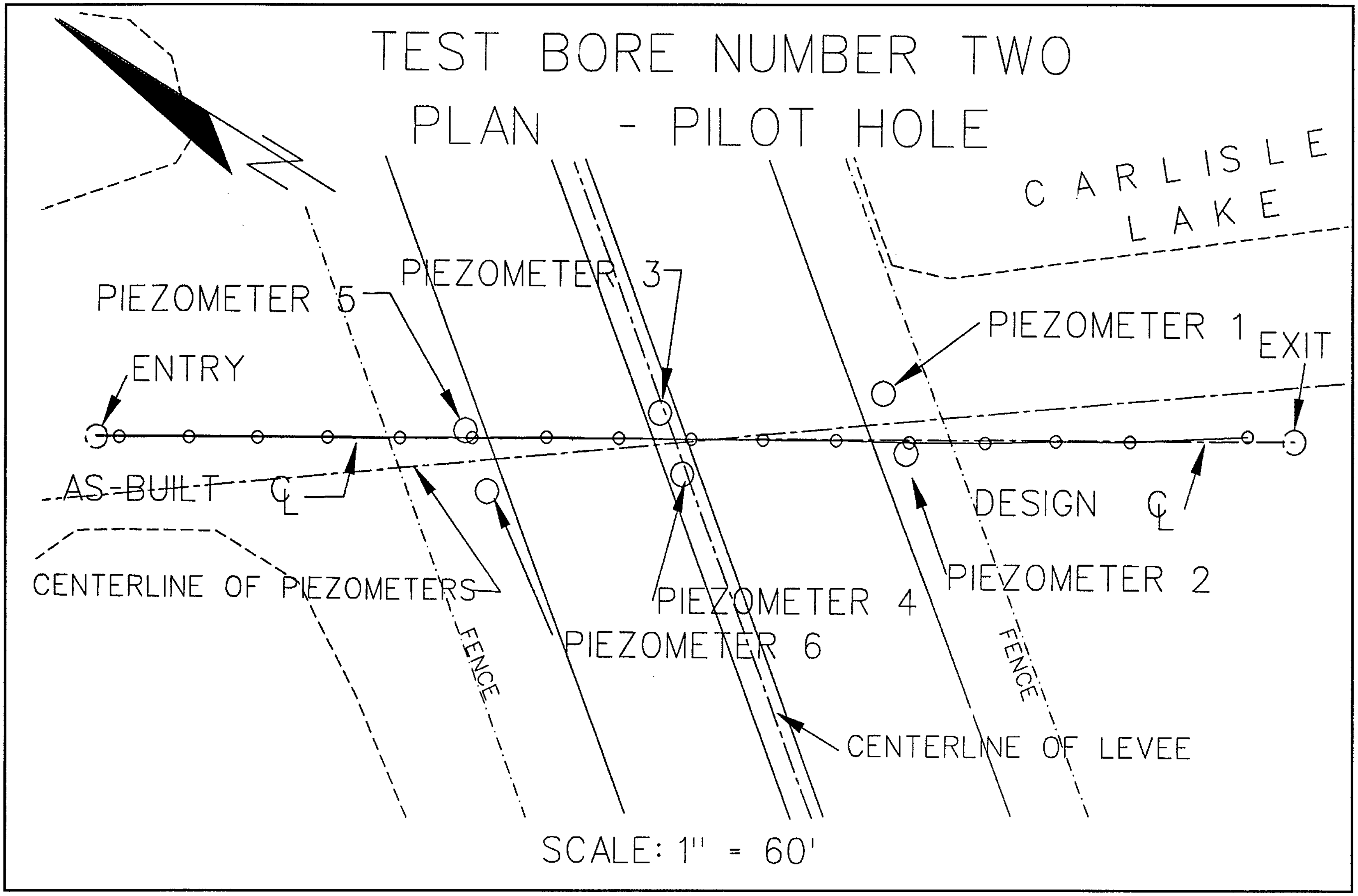


Figure 27. Plan view of Bore 2; planned and as-built location. (To convert inches to centimeters, multiply by 2.54; to convert feet to meters, multiply by 0.305)

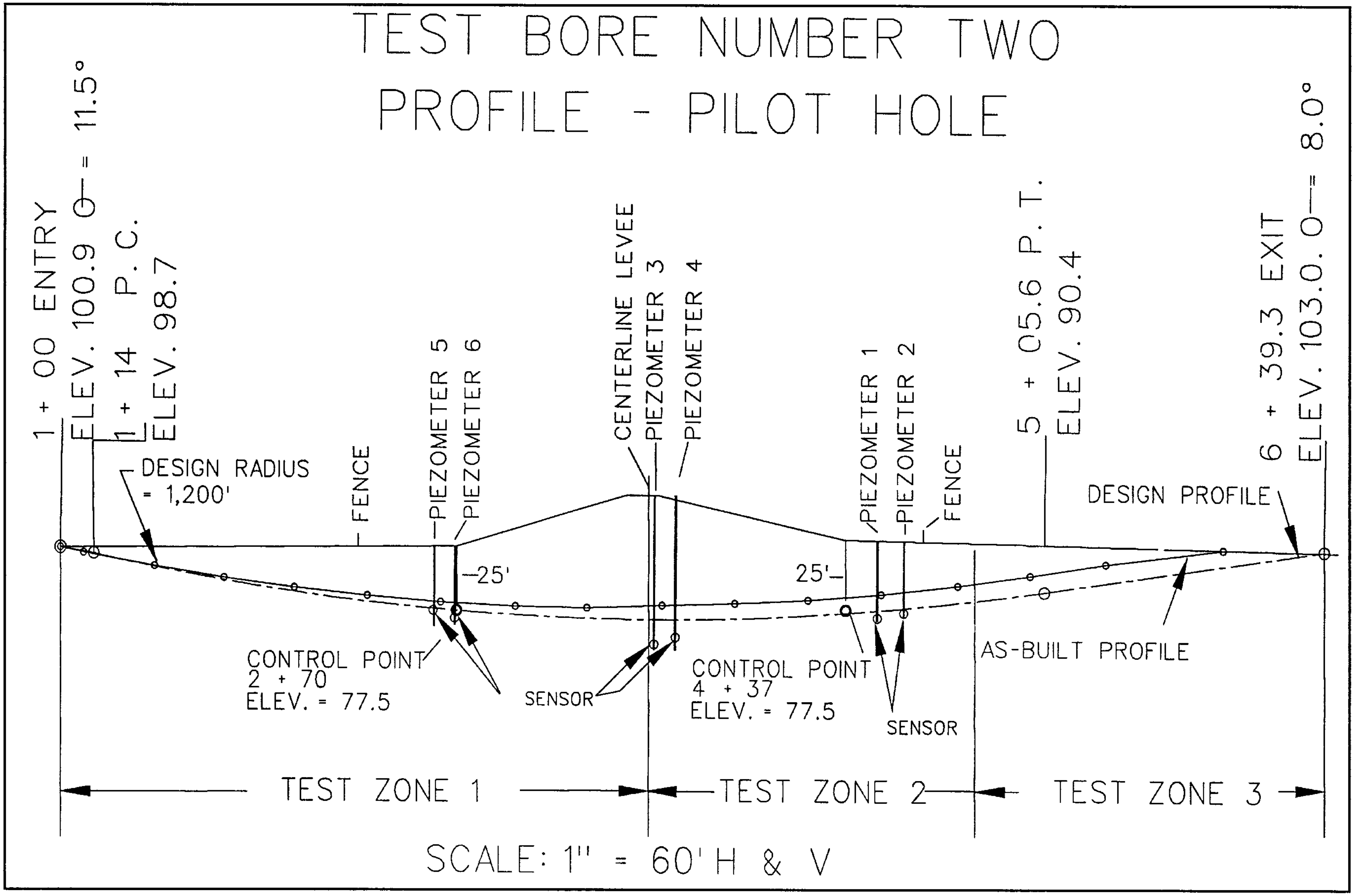


Figure 28. Profile view of Test Bore 2; planned and as-built location. (To convert inches to centimeters, multiply by 2.54; to convert feet to meters, multiply by 0.305)

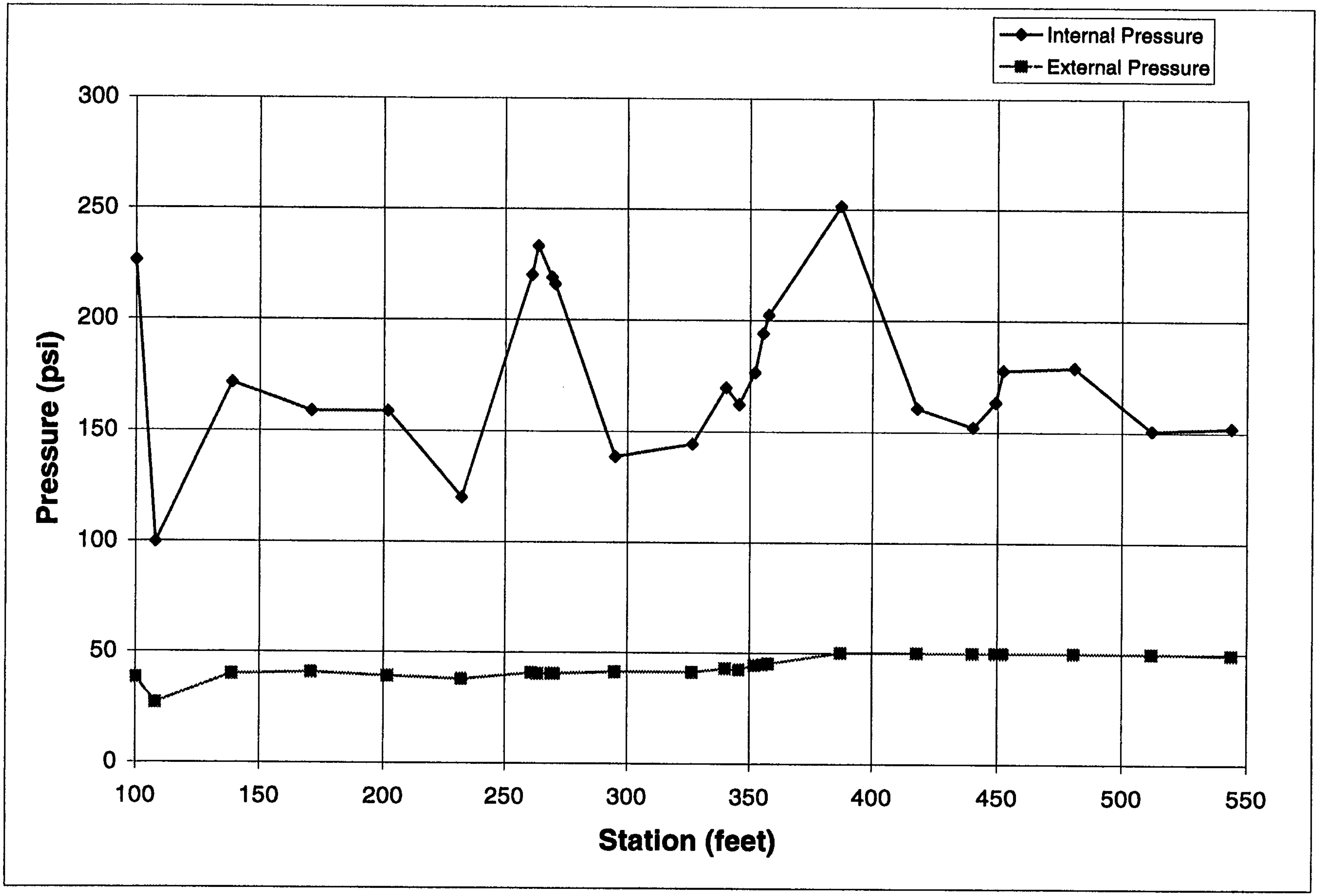


Figure 29. Internal and external pressures measured during Pilot Bore 2. (To convert psi to kN/m^2, multiply by 6.89)

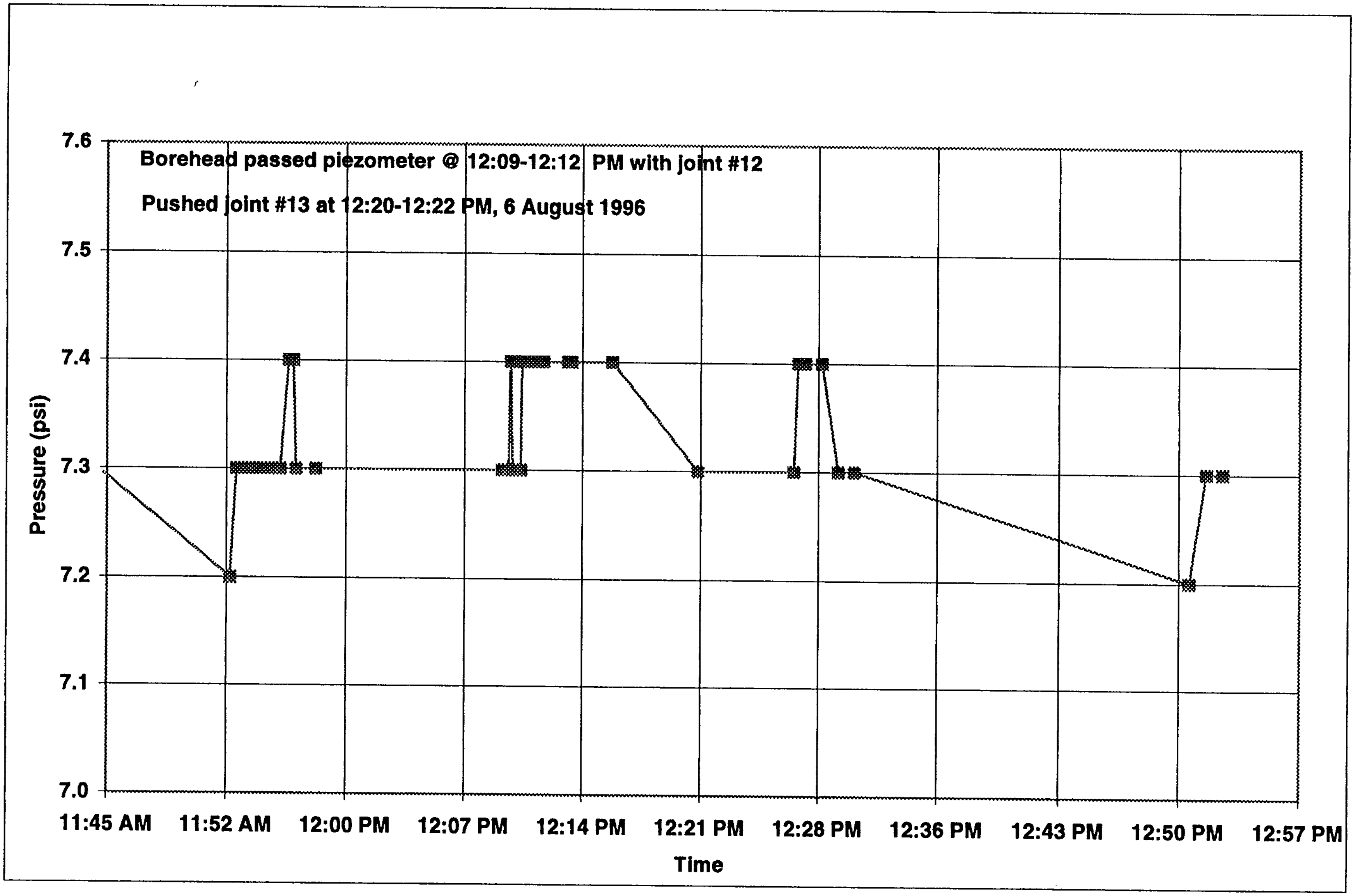


Figure 30. Piezometer 1, Pilot Bore 2. (To convert psi to kN/m^2, multiply by 6.89)

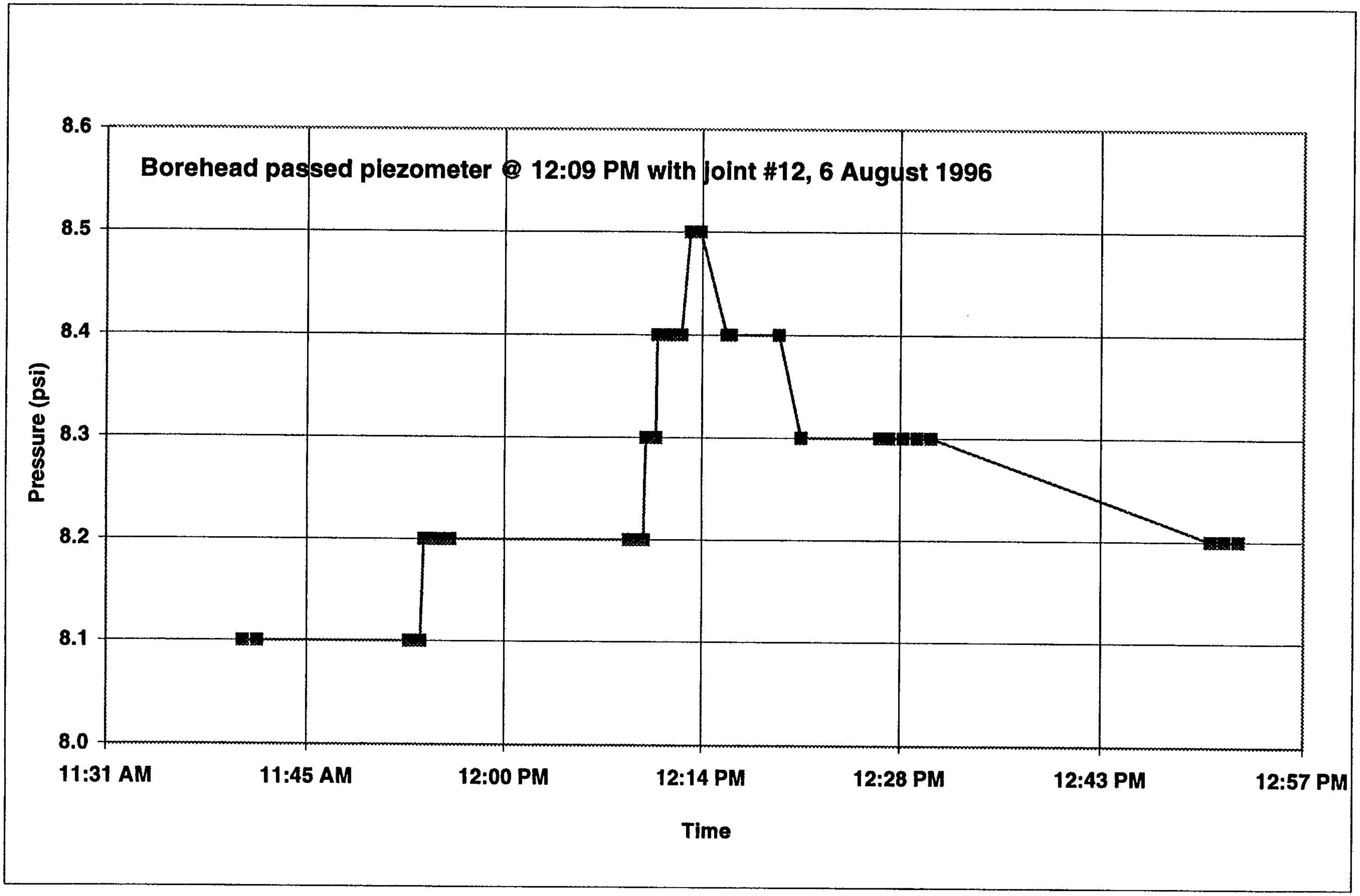


Figure 31. Piezometer 2, Pilot Bore 2. (To convert psi to kN/m^2, multiply by 6.89)

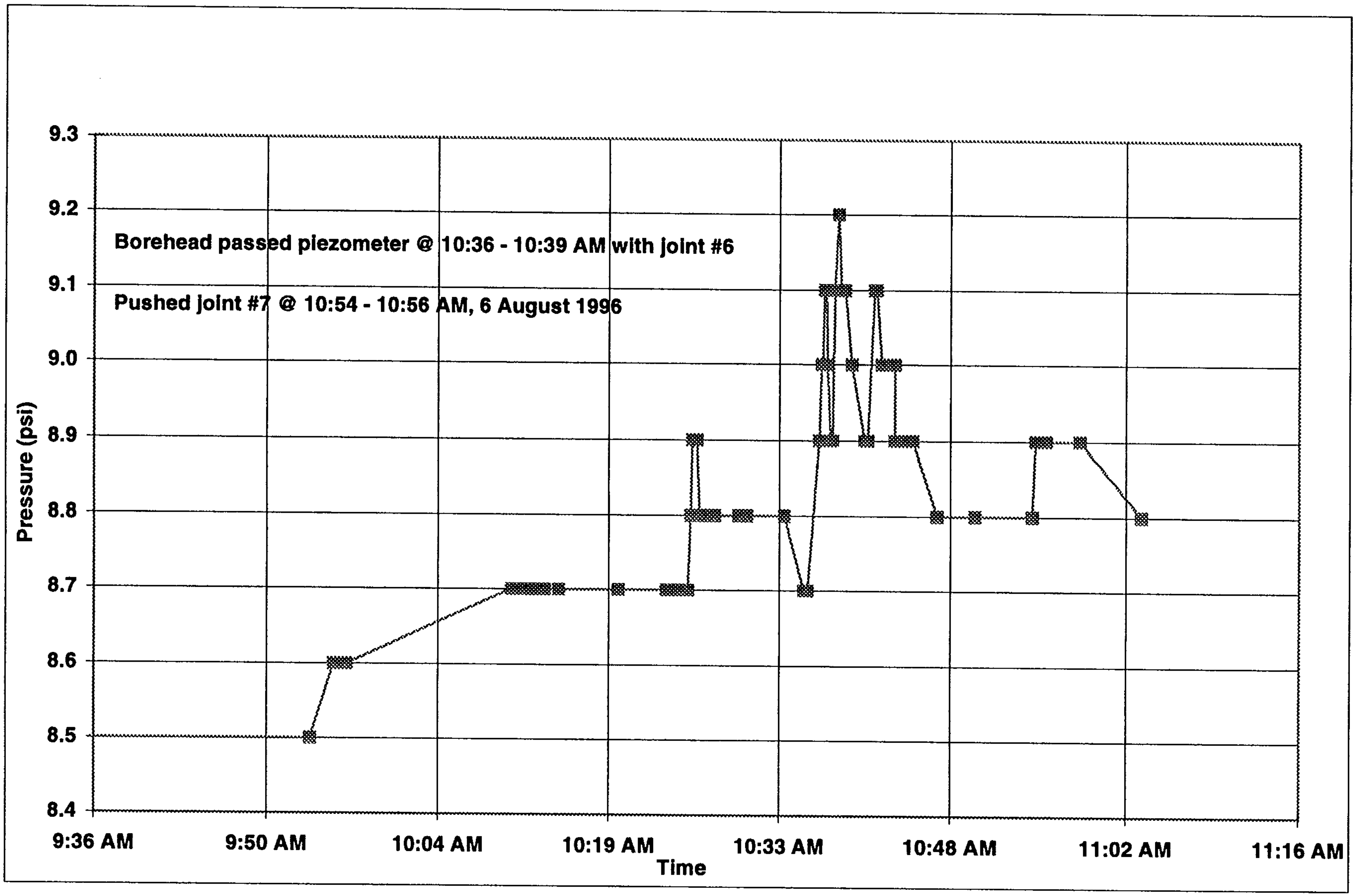


Figure 32. Piezometer 5, Pilot Bore 2. (To convert psi to kN/m^2, multiply by 6.89)

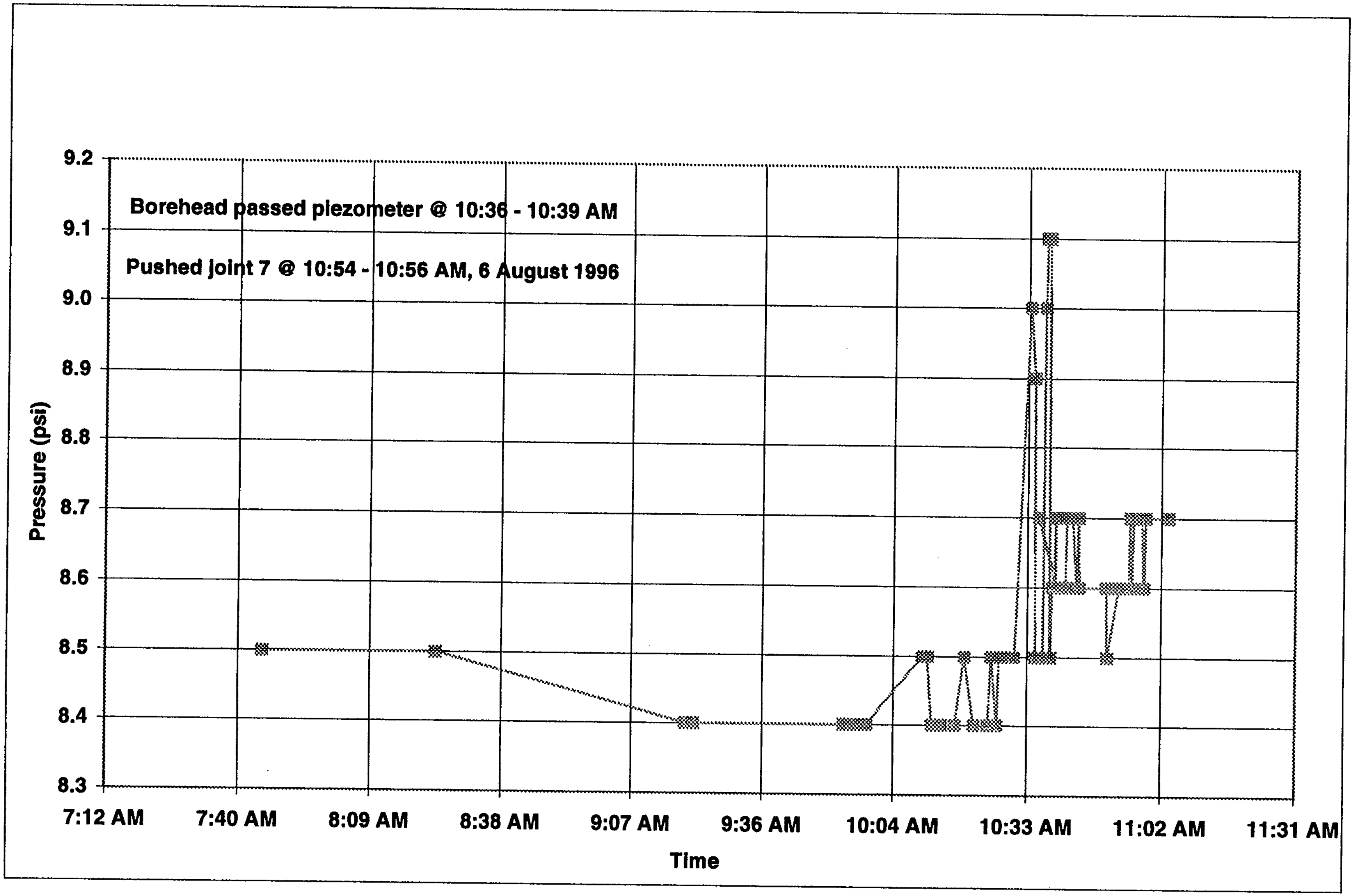


Figure 33. Piezometer 6, Pilot Bore 2. (To convert psi to kN/m², multiply by 6.89)

Figure 34. Barrel reamer and swivel assembly

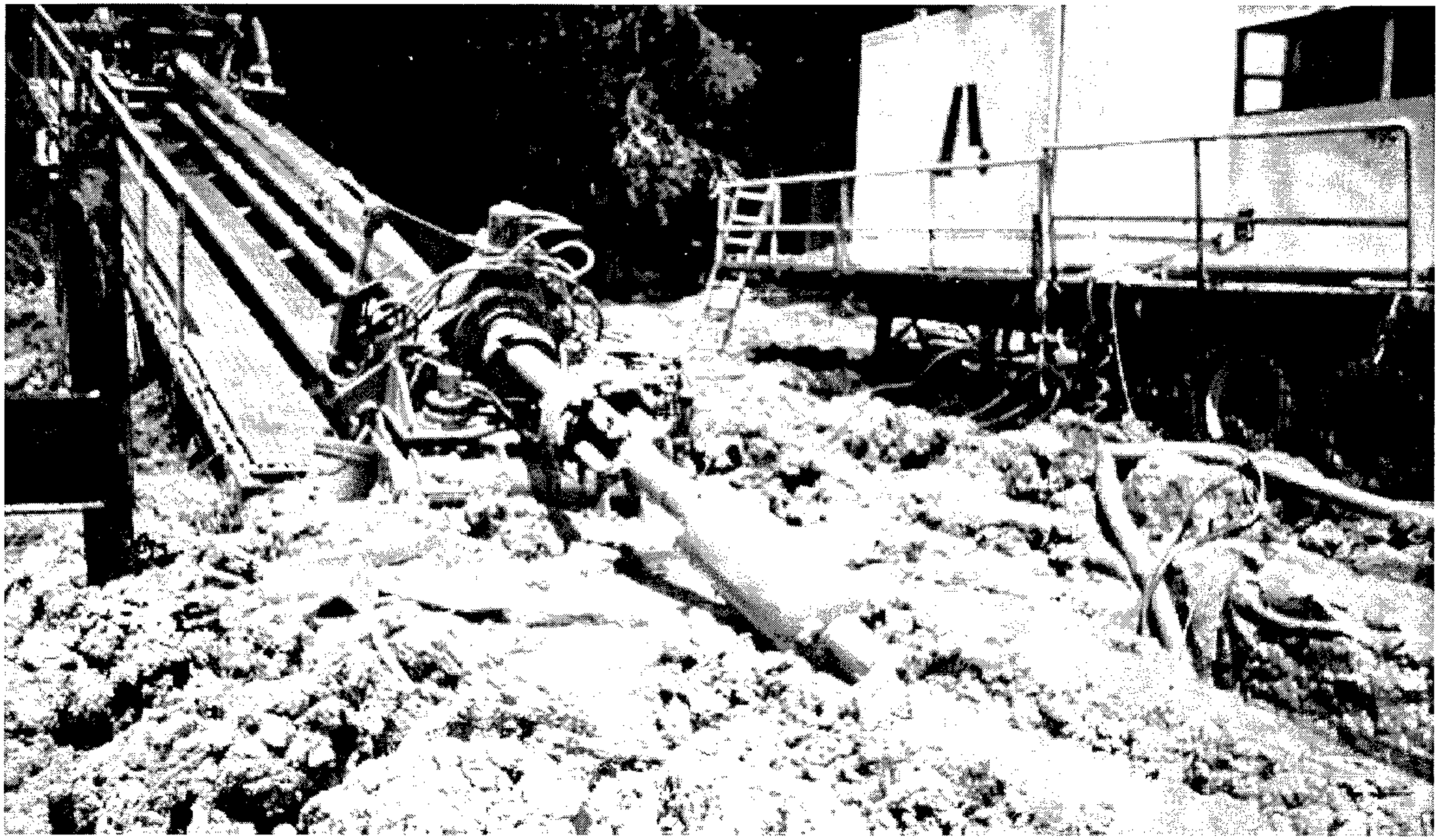


Figure 35. Completion of reaming on Bore 2

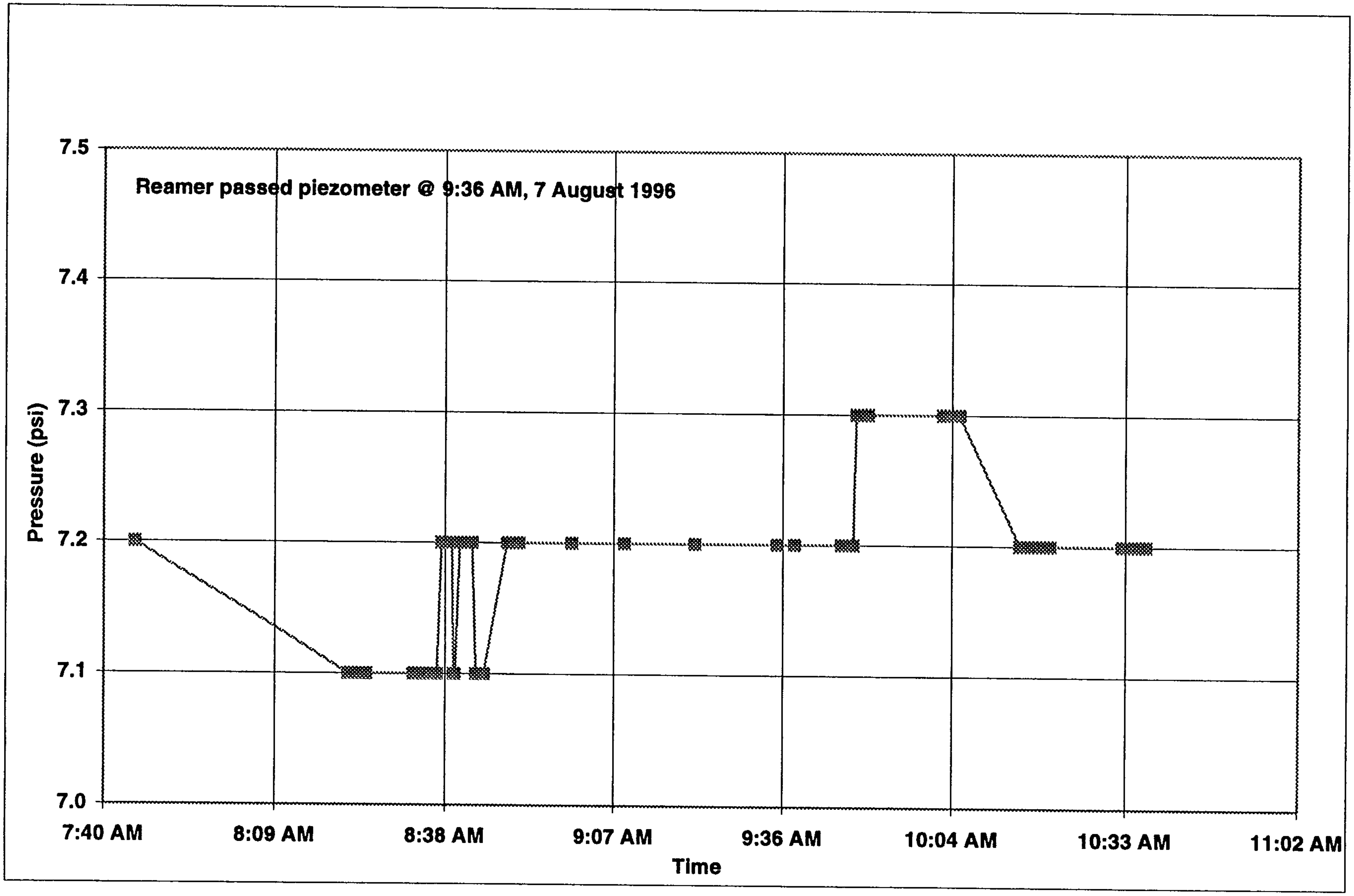


Figure 36. Piezometer 1, Reaming Bore 2. (To convert psi to kN/m^2, multiply by 6.89)

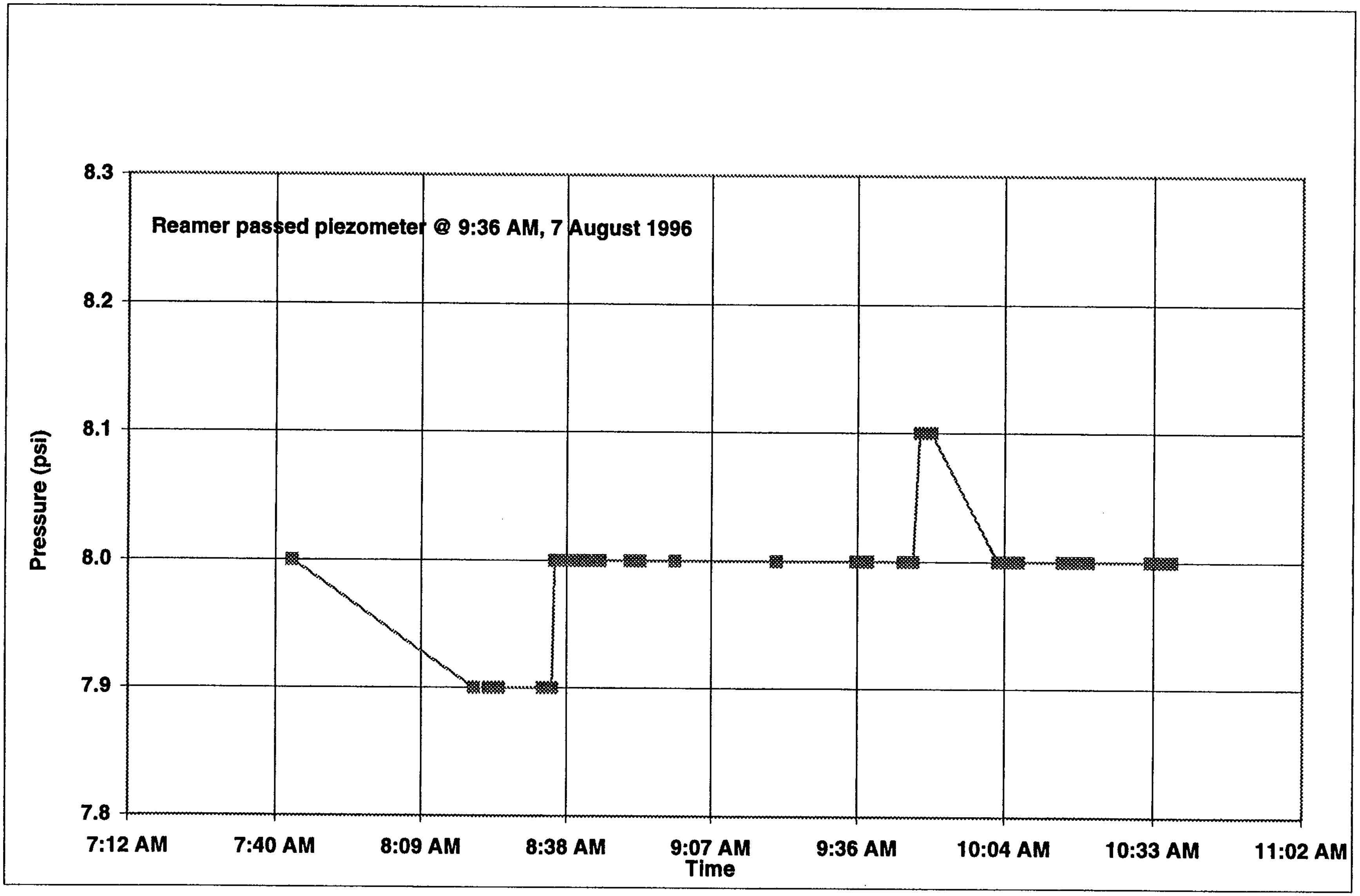


Figure 37. Piezometer 2, Reaming Bore 2. (To convert psi to kN/m^2, multiply by 6.89)

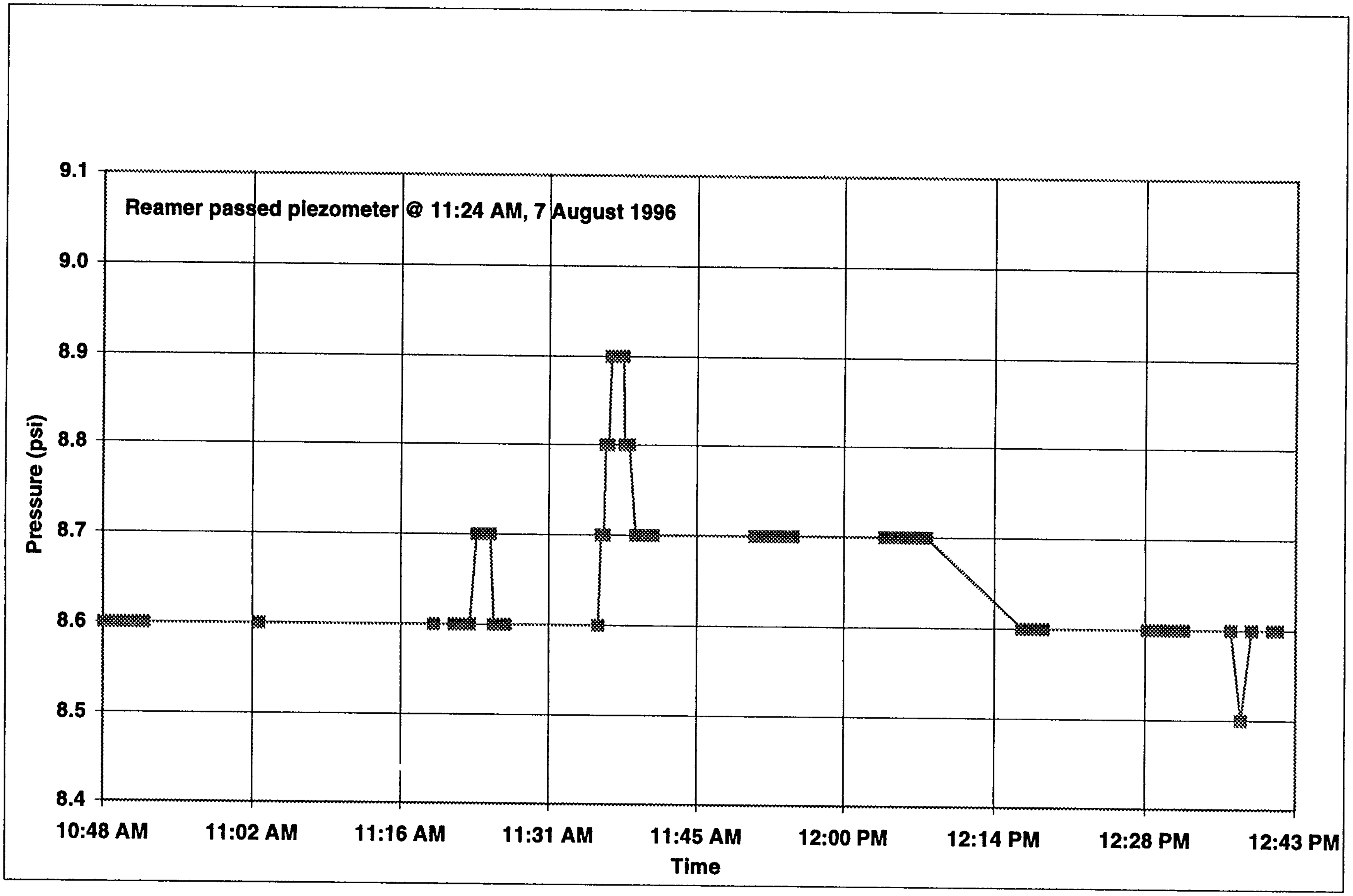


Figure 38. Piezometer 5, Reaming Bore 2. (To convert psi to kN/m^2, multiply by 6.89)

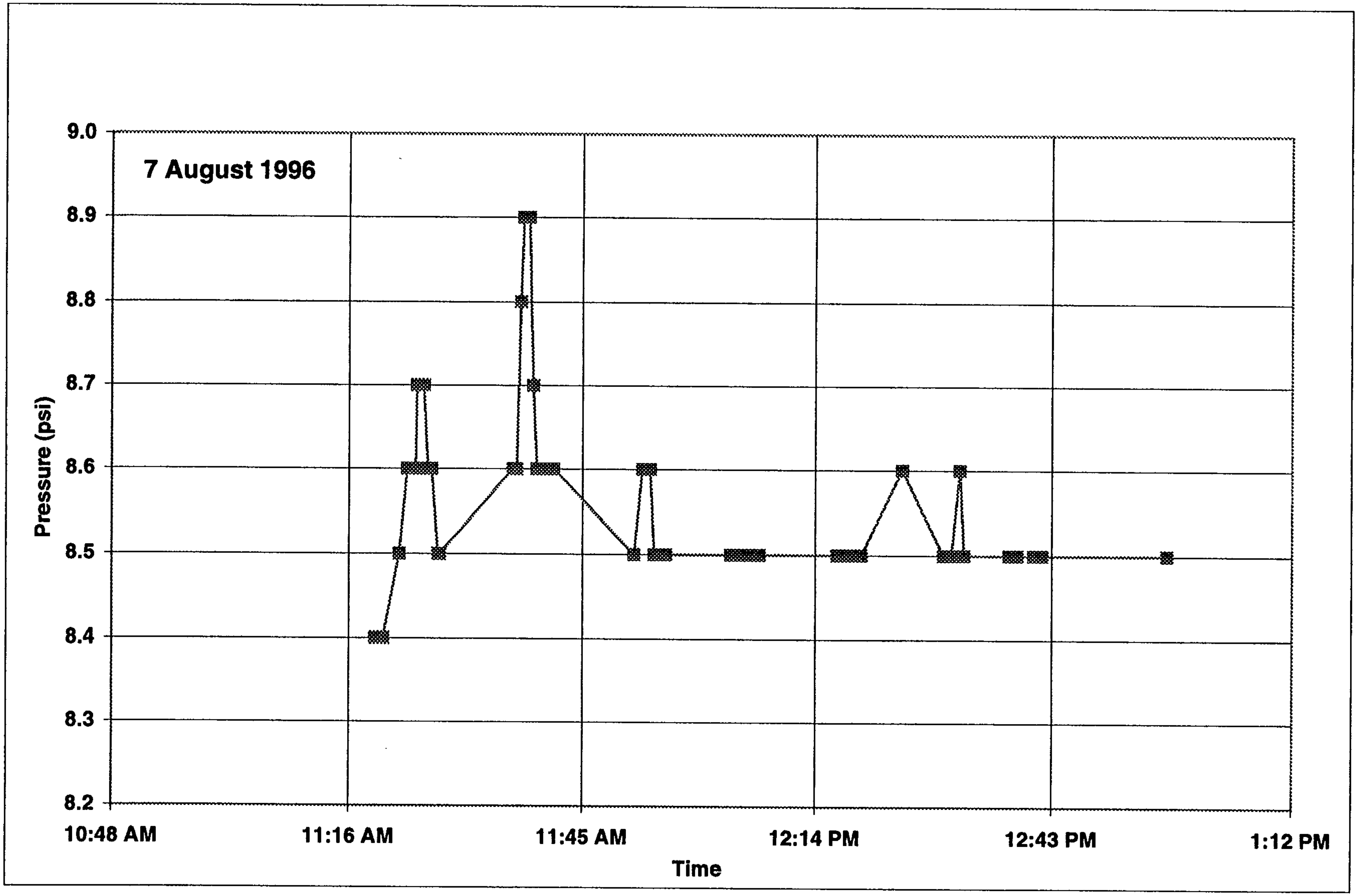


Figure 39. Piezometer 6, Reaming Bore 2. (To convert psi to kN/m^2, multiply by 6.89)

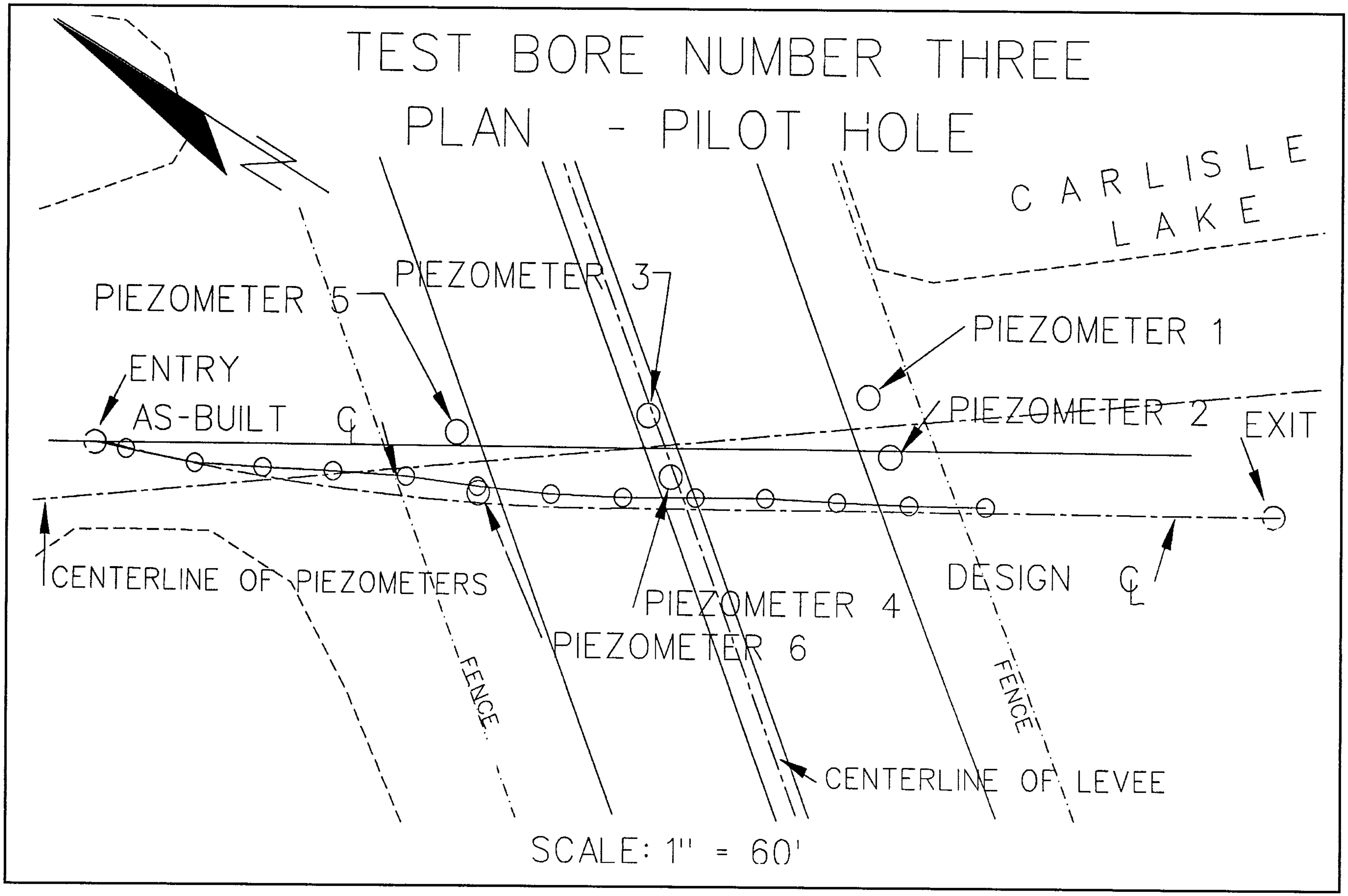


Figure 40. Plan view of Bore 3; planned and as-built location. (To convert feet to meters, multiply by 0.305; to convert inches to centimeters, multiply by 2.54)

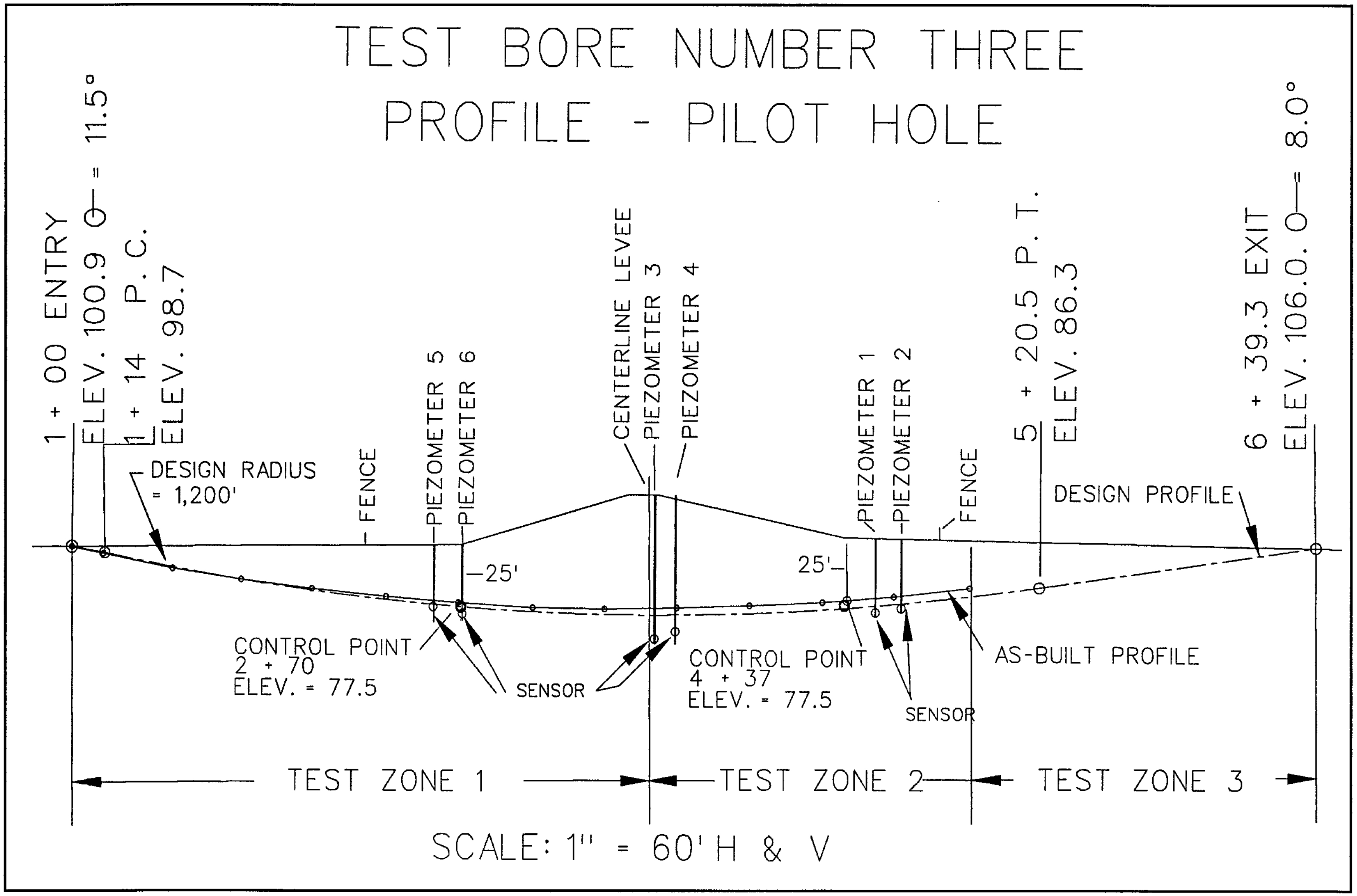


Figure 41. Profile view of Bore 3; planned and as-built location. (To convert feet to meters, multiply by 0.305; to convert inches to centimeters, multiply by 2.54; to convert degrees to radians, multiply by 0.017)

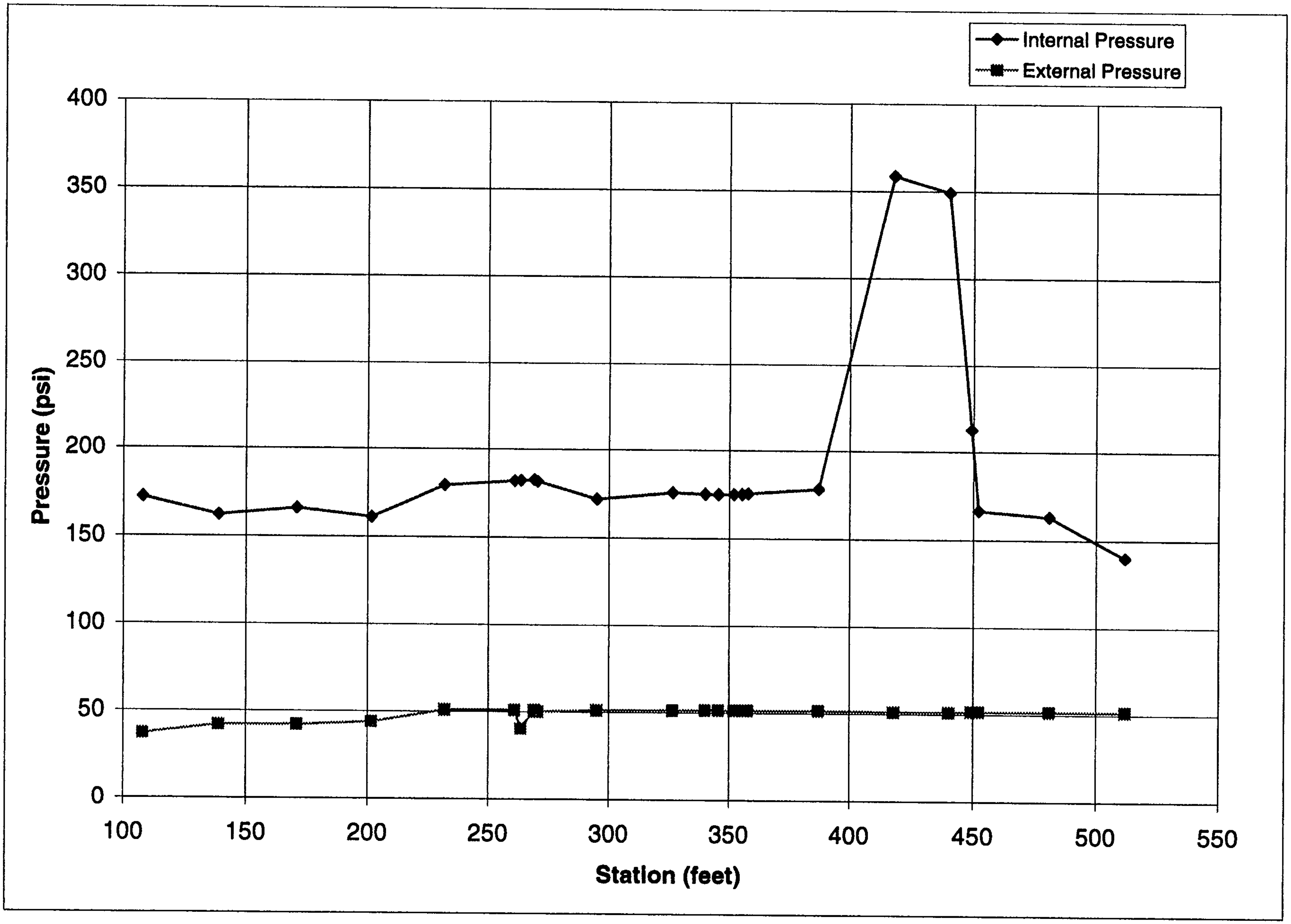


Figure 42. Internal and external pressures measured during Pilot Bore 3. (To convert psi to kN/m^2, multiply by 6.89)

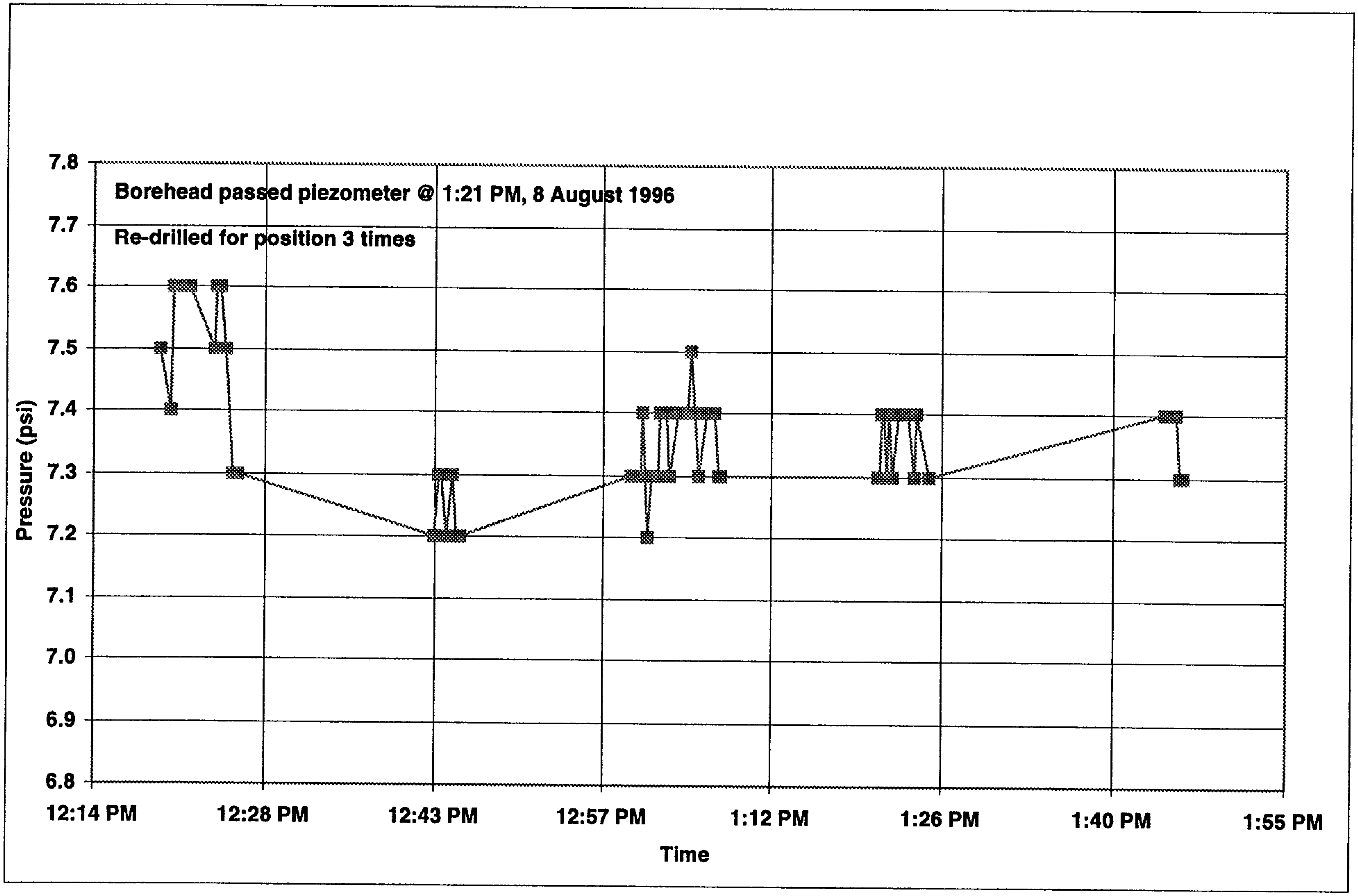


Figure 43. Piezometer 1, Pilot Bore 3. (To convert psi to kN/m^2, multiply by 6.89)

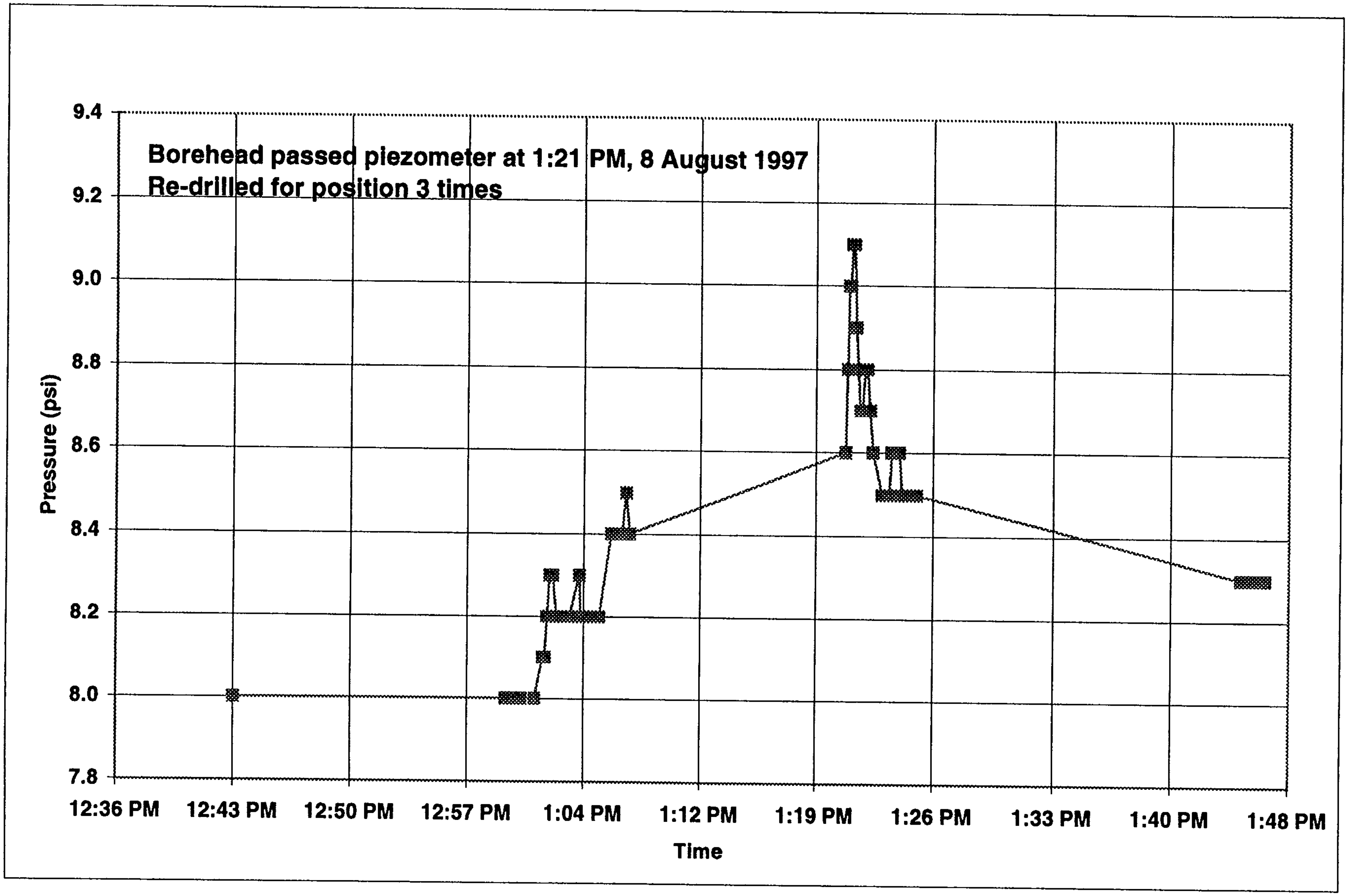


Figure 44. Piezometer 2, Pilot Bore 3. (To convert psi to kN/m^2, multiply by 6.89)

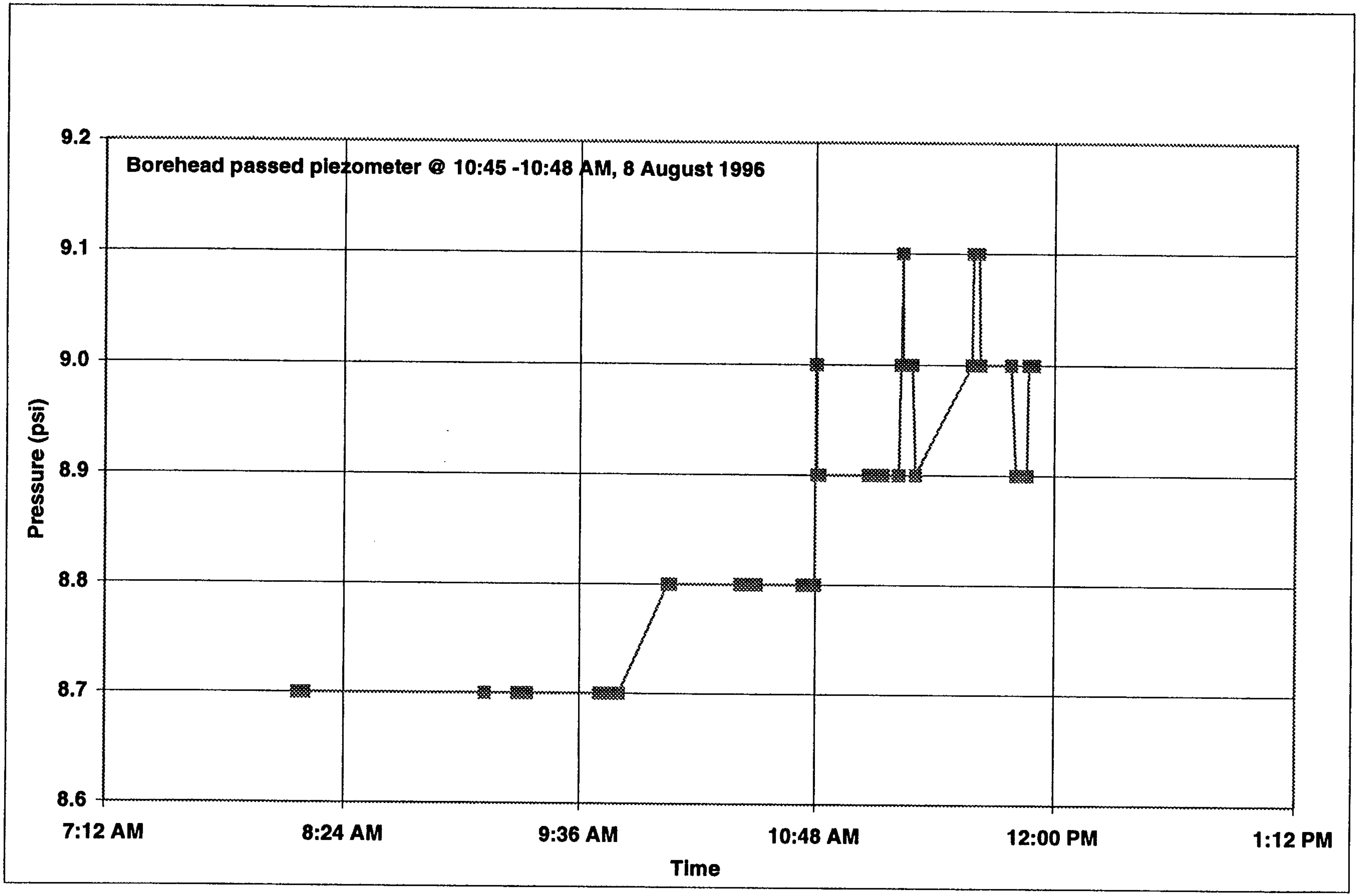


Figure 45. Piezometer 5, Pilot Bore 3. (To convert psi to kN/m^2, multiply by 6.89)

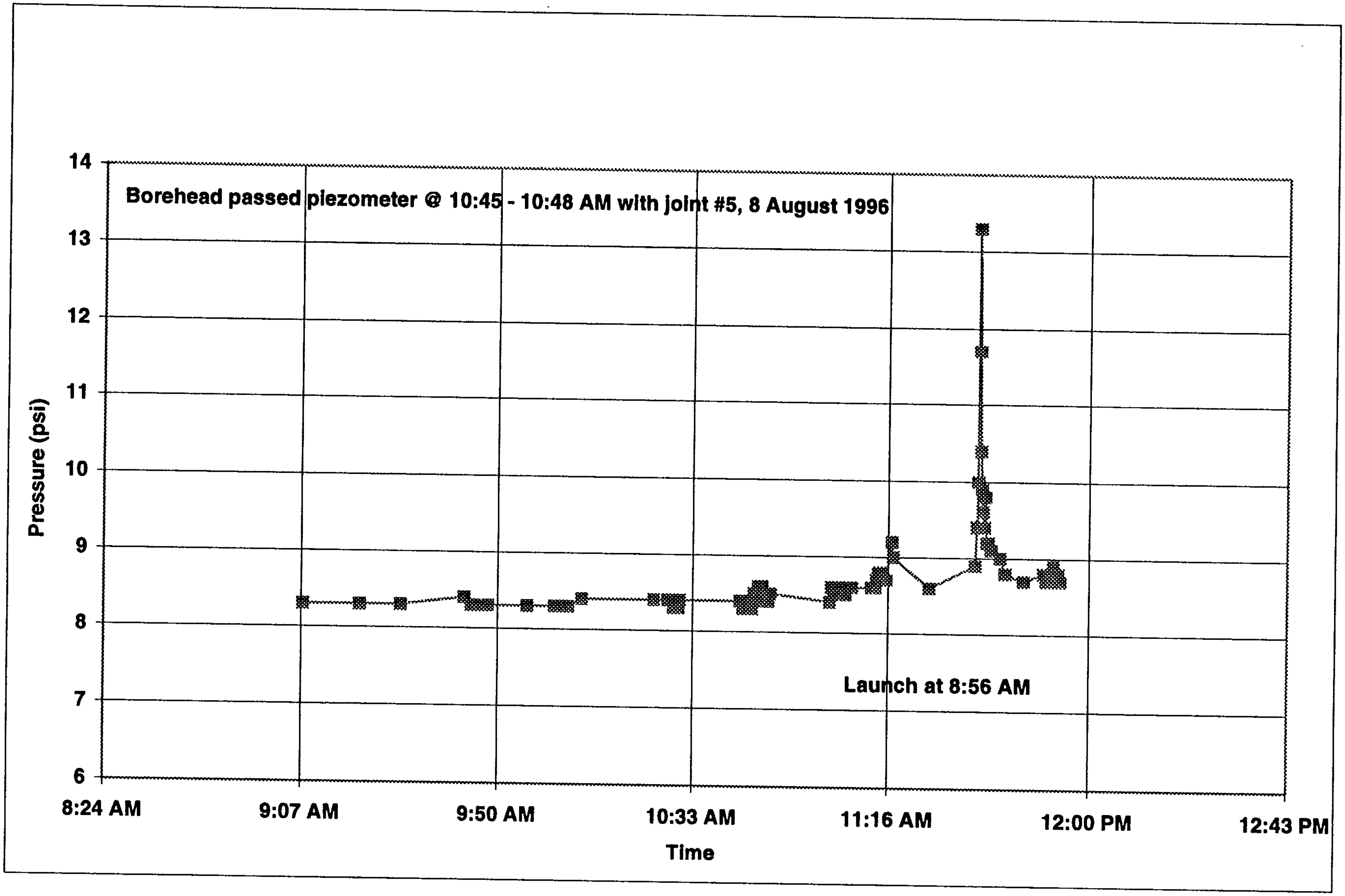


Figure 46. Piezometer 6, Pilot Bore 3. (To convert psi to kN/m^2, multiply by 6.89)

Figure 47. Autopsy on land side of levee

Figure 48. Exposed Bores 1 and 2 on land side of levee

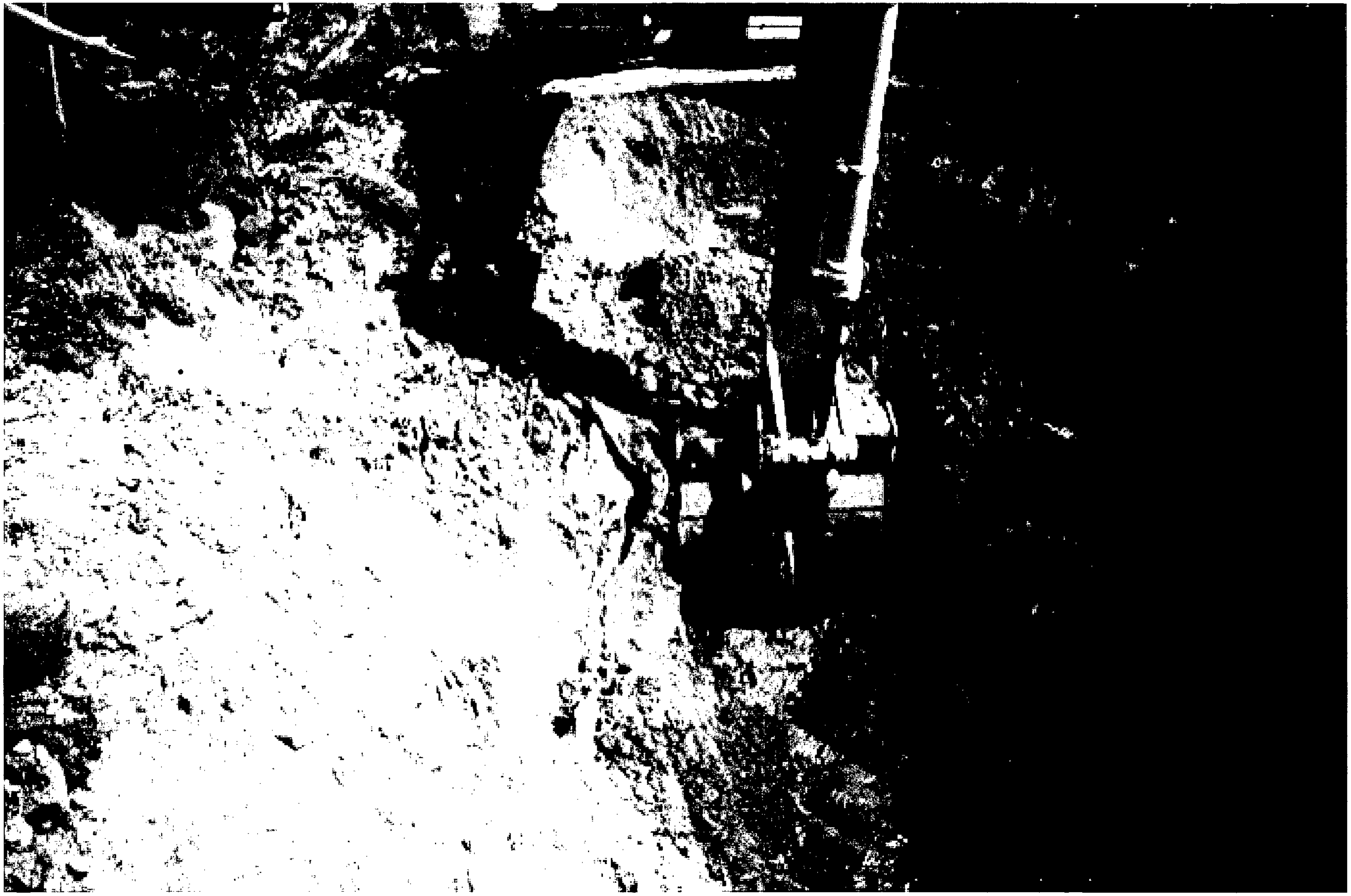


Figure 49. Borehole below the water table

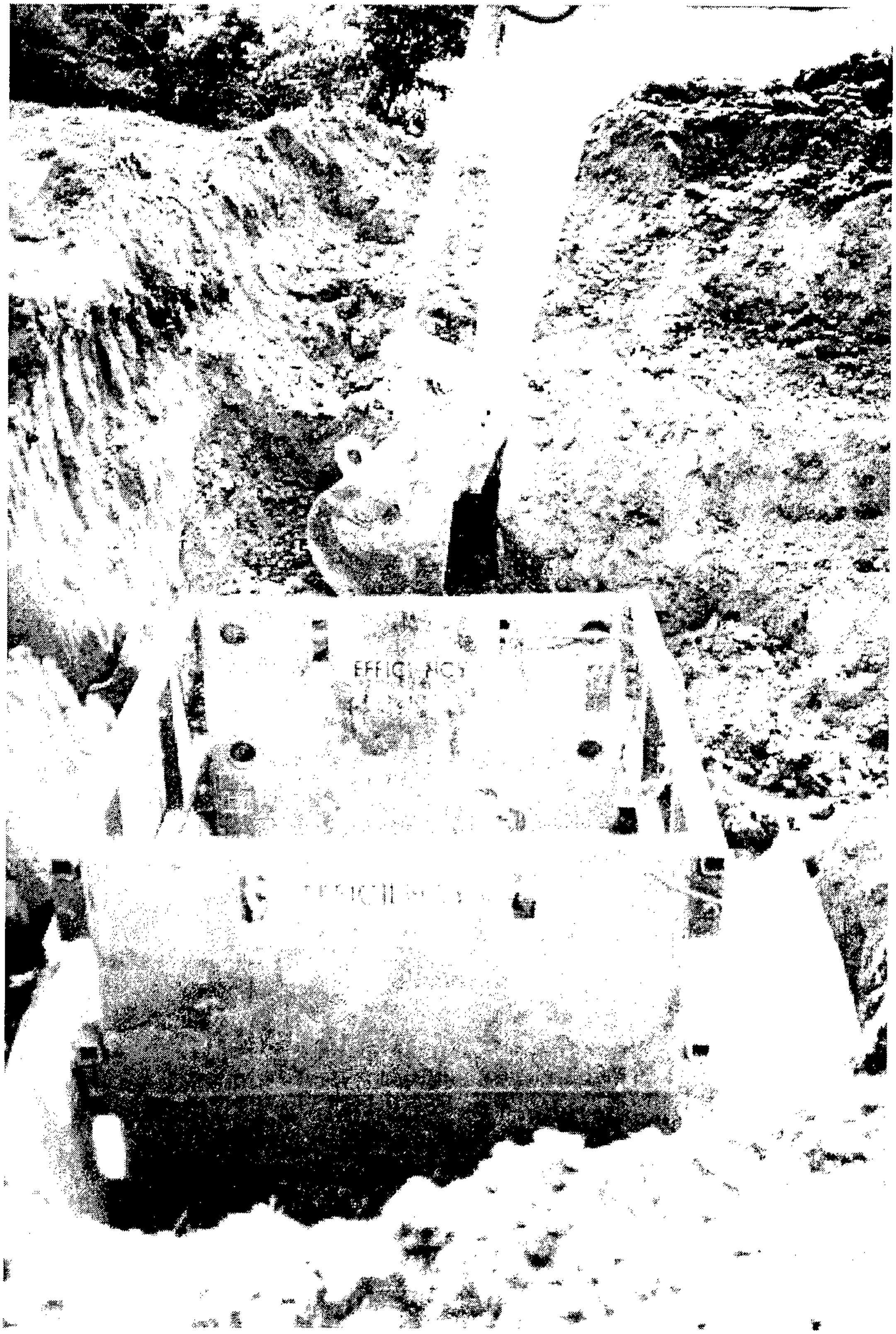


Figure 50. Trench box used for excavation below water table

Figure 51. Hand excavation below water table

Figure 52. Locating borehole with track-hoe

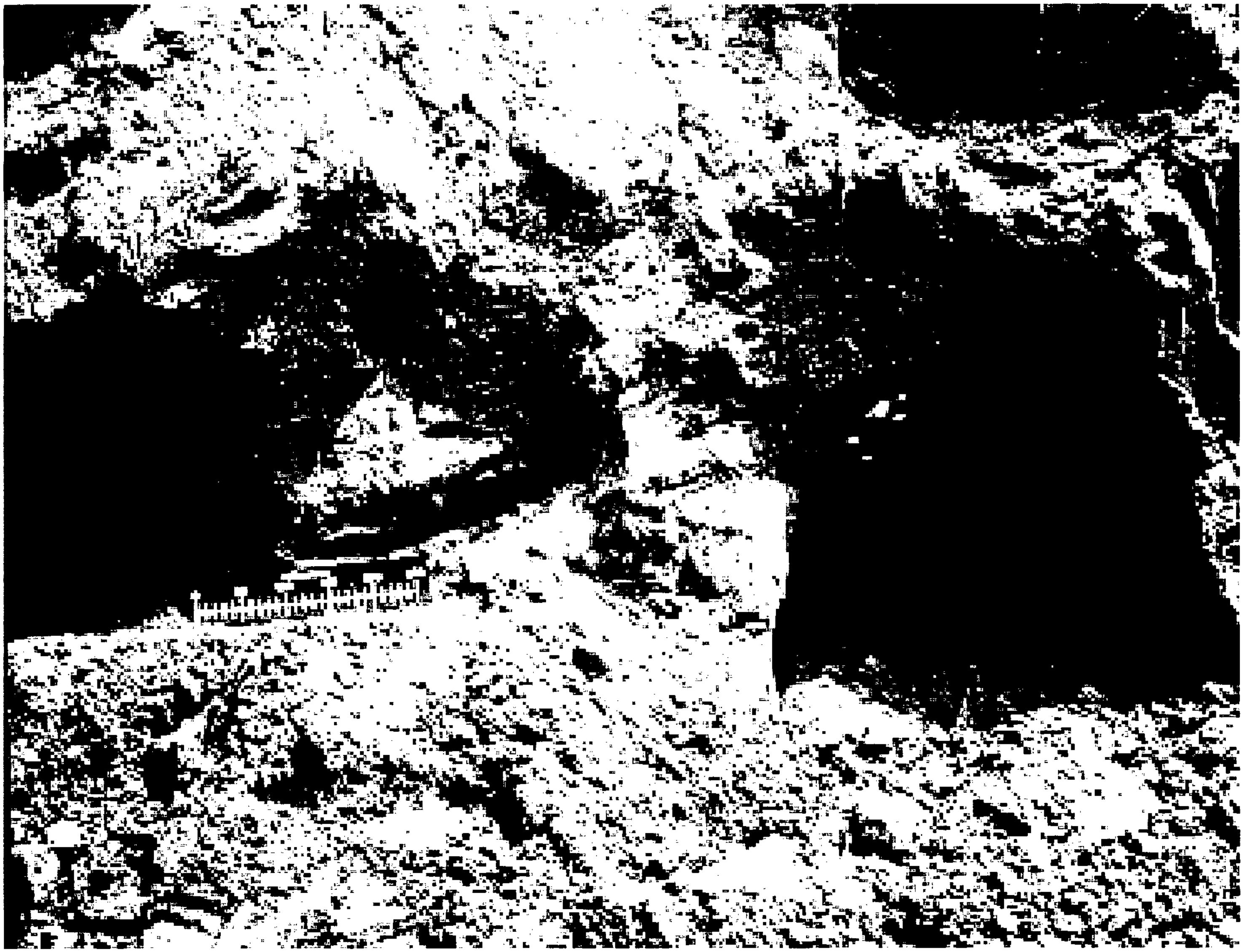


Figure 53. Bore 1, landside

Figure 54. Horizontal hydrofracture, Bore 1, landside

Figure 55. Bore 1 riverside - collapsed hole

Figure 56. Measuring Bore 1, landside

Figure 57. Bore 2, landside

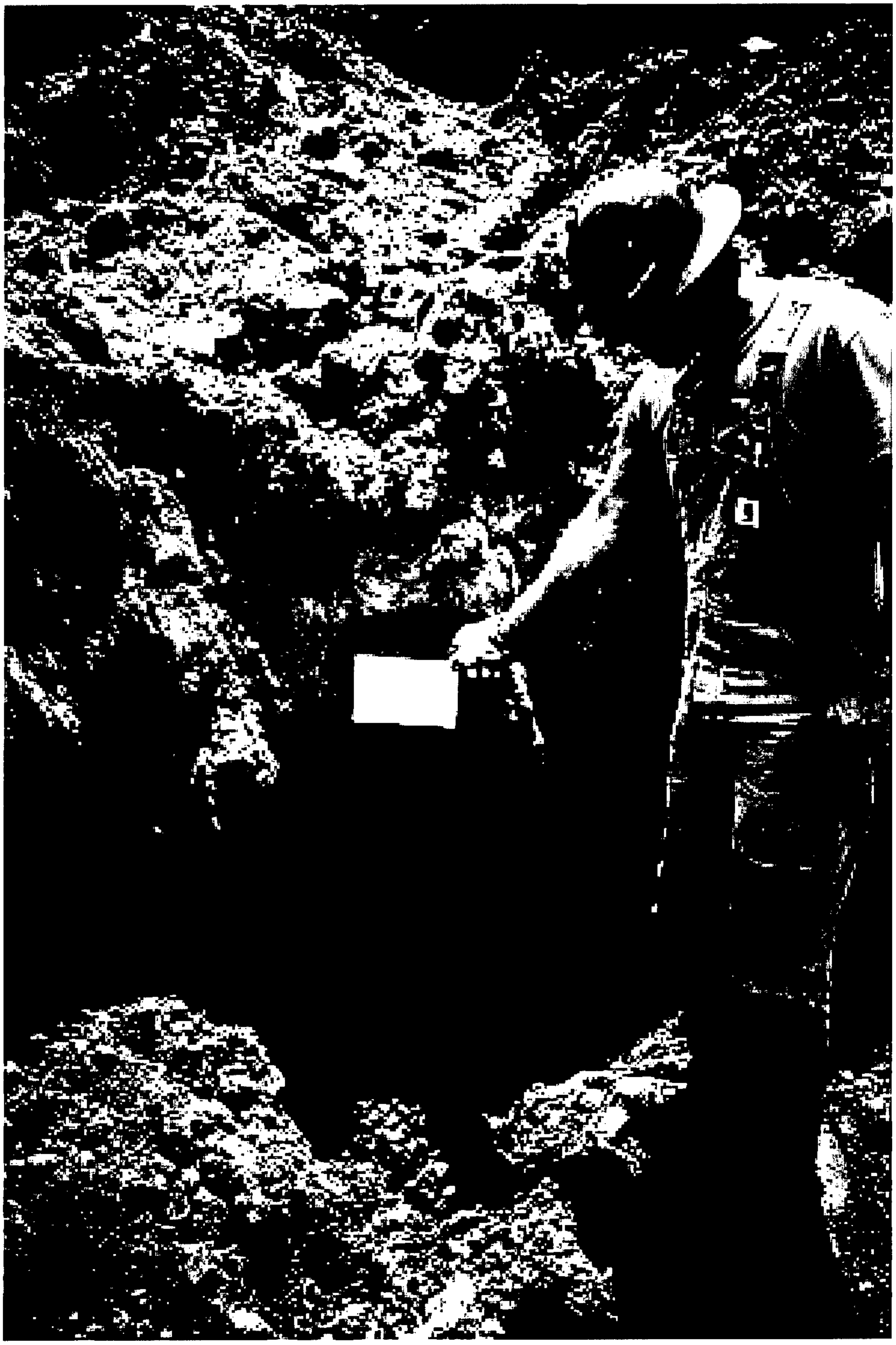


Figure 58. Vertical crack above Bore 2, landside

Figure 59. Exposed vertical crack

Figure 60. Close-up of the vertical crack - Bore 2, landside

Figure 61. Bore 2 exposed on riverside

Figure 62. Drilling fluid exposed - Bore 2, Riverside

Figure 63. Pilot Bore 3 identification

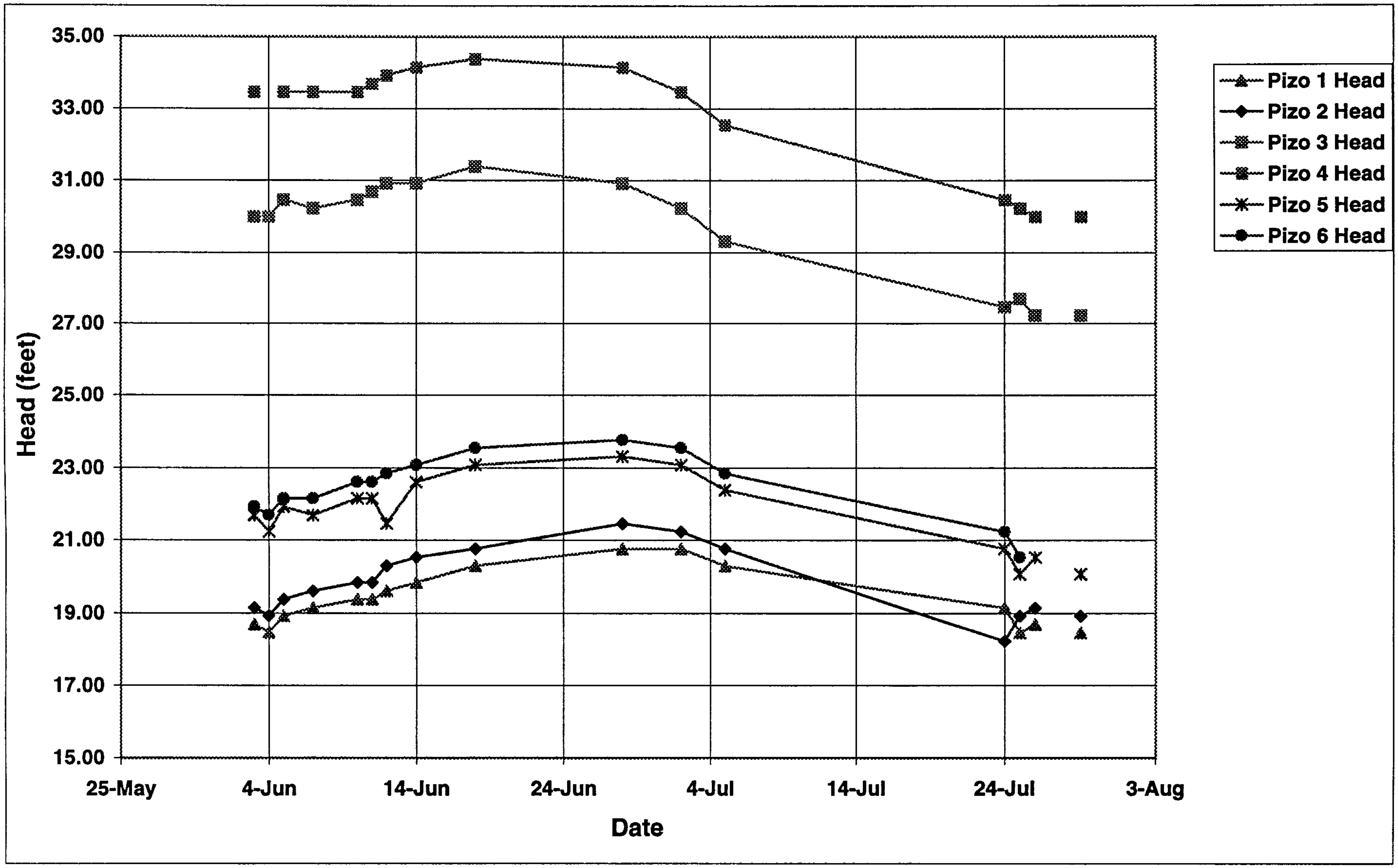


Figure 64. Piezometer readings reflecting changes in the Mississippi River levels prior to the test. To convert feet to meters, multiply by 0.305)

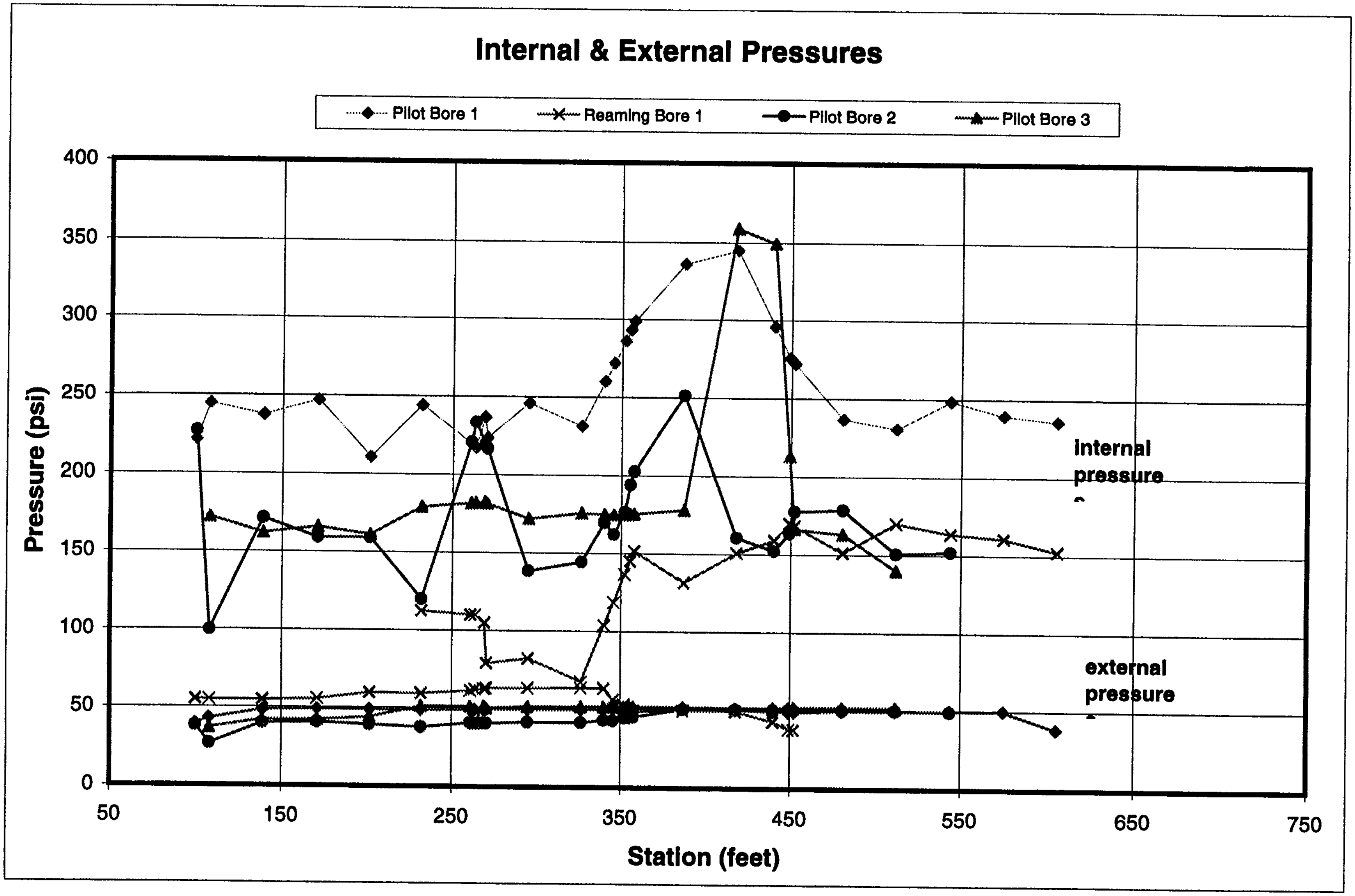


Figure 65. Internal and external pressures on Bores 1 through 3. (To convert feet to meters, multiply by 0.305; to convert psi to kN/m^2, multiply by 6.89)

Appendix A Recommended Guidelines for Installation of Pipelines Beneath Levees Using Horizontal Directional Drilling

The guidelines for the installation of pipelines using horizontal directional drilling are based on the results and conclusions of the field evaluation of the Construction Productivity Advancement Research (CPAR) project, Installation of Pipelines Beneath Levees using Horizontal Directional Drilling (HDD), as well as analytical studies of soil/drilling fluid interaction and evaluation of case history data. The recommended guidelines are appropriate for projects on which the Corps of Engineers (CE) has jurisdiction and may be used for non-CE projects, as well. The recommendations address the main issues of concern that have been expressed by CE District personnel, either in District regulations or in meetings and discussions. In addition to these recommendations, the reader should consult the references listed in Chapter 1 of the main body of this report. References are also listed at the end of this Appendix.

With each proposed crossing, it is important that the critical elements of the HDD process are addressed to enhance the possibilities of a successful levee crossing. Addressing each element will greatly reduce the risk of creating preferential seepage paths or other phenomena that threaten the stability of the levee. Key elements addressed in this guideline include:

a. Establishing allowable drilling fluid pressures.

b. Monitoring drilling fluid pressures.

c. Establishing appropriate setback distances.

d. Establishing appropriate depths of cover over the pipeline.

e. Controlling speed of drilling.

f. Evaluating effects of groundwater.

g. Prevention of seepage and erosion.

h. Use of closure devices.

i. Use of relief wells.

When establishing the appropriate parameters for each project, it is important to have accurate geotechnical information. Many of the key parameters for a project, including limiting pressures, setback distances, and depth of cover, depend on soil properties and geotechnical data gathered during preconstruction geotechnical investigations or collected during construction of the levee. Without accurate soil investigation data, it will be difficult to determine appropriate drilling parameters and could result in inappropriate design.

Allowable Drilling Fluid Pressures

There are legitimate concerns associated with the fluid pressures used for excavation during the horizontal directional drilling process and the risk of hydraulic fracturing. Reasonable limits must be placed on maximum fluid pressures in the annular space of the bore to prevent inadvertent drilling fluid returns to the ground surface. However, it is equally important that drilling pressures remain sufficiently high to maintain borehole stability, since the ease in which the pipe will be inserted into the borehole is dependent upon borehole stability. Limiting borehole pressures are a function of pore pressure, the pressure required to counterbalance the effective normal stresses acting around the bore (depth), and the undrained shear strength of the soil.

Maximum allowable mud pressures

To establish the maximum allowable mud pressure, Delft Geotechnics (1997)[1] has suggested use of the following equation which is based on cavity expansion theory (Appendix B):

$$P_{\lim} = \left(P_f + c \cdot \cot\phi\right)(Q)^{\frac{-\sin\phi}{1+\sin\phi}} \left\{\left(\frac{R_o}{R_{p,\max}}\right)^2 + Q\right\}^{\frac{-\sin\phi}{1+\sin\phi}} - c \cdot \cot\phi \qquad \text{(A1)}$$

where

$P_{\lim}$ = limiting mud pressure

P_f = mud pressure at onset of plastic failure

$$P_f = \sigma'_0(1+\sin\phi)+c(\cos\phi)$$

[1] Reference sources are listed at the end of this Appendix.

σ'_0 = initial effective stress

c = effective cohesion

ϕ = effective internal angle of friction

Q = a function of the shear modulus and effective stress

$$Q = \frac{\sigma'_0(\sin\phi) + c(\cos\phi)}{G}$$

G = shear modulus

R_0 = initial radius of the borehole

$R_{p,\max}$ = radius of the plastic zone

However, the cavity expansion theory is based on an infinite plastic zone. The equation given by Delft Geotechnics depends on the determination of a "safe radius" ($R_{p,\max}$) around the borehole in which the drilling mud will remain, also referred to as the maximum allowable radius of the plastic zone. The equation determines the pressure that would cause drilling fluid to exit the maximum radius of the plastic zone. For the determination of the maximum radius of the plastic zone, Delft Geotechnics suggests using a value of H/2 for clay soils and 2/3 H for sandy soils, where H represents the height of soil cover over the pipeline. Using this equation along with values for the internal angle of friction, the shear modulus of the soil, and the initial pore pressure, the maximum allowable mud pressure can be determined over the length of the bore.

Figure A1 shows limiting mud pressures as a function of depth for a typical sand and soft clay. For these calculations, it was assumed that the water table was located at the ground surface. The values used in the calculation for limiting pressure are listed in Table A1.

From Figure A1, it is easily seen that with the cavity expansion theory the limiting pressures in a sandy material are much higher than in a clay material, except for very shallow depths. This is largely due to the frictional properties exhibited by the sand which inhibits cavity expansion.

For the CPAR test, a report of the contract geotechnical investigation is presented in Appendix C. For the six borings, the following tests were made on selected soil samples: wet and dry unit weight, unconfined compressive strength, Atterberg limits, moisture contents, and sieve tests.

Figure A2 shows the maximum allowable mud pressures determined for the CPAR field test. From this figure it is easily seen that the maximum allowable pressure varies with the depth of soil cover. Based on these calculations, it would be necessary for the pressure in the annular space of the bore to remain below the maximum allowable pressure throughout the drilling process to minimize the potential for initiating plastic yield and losing drilling mud to the surface.

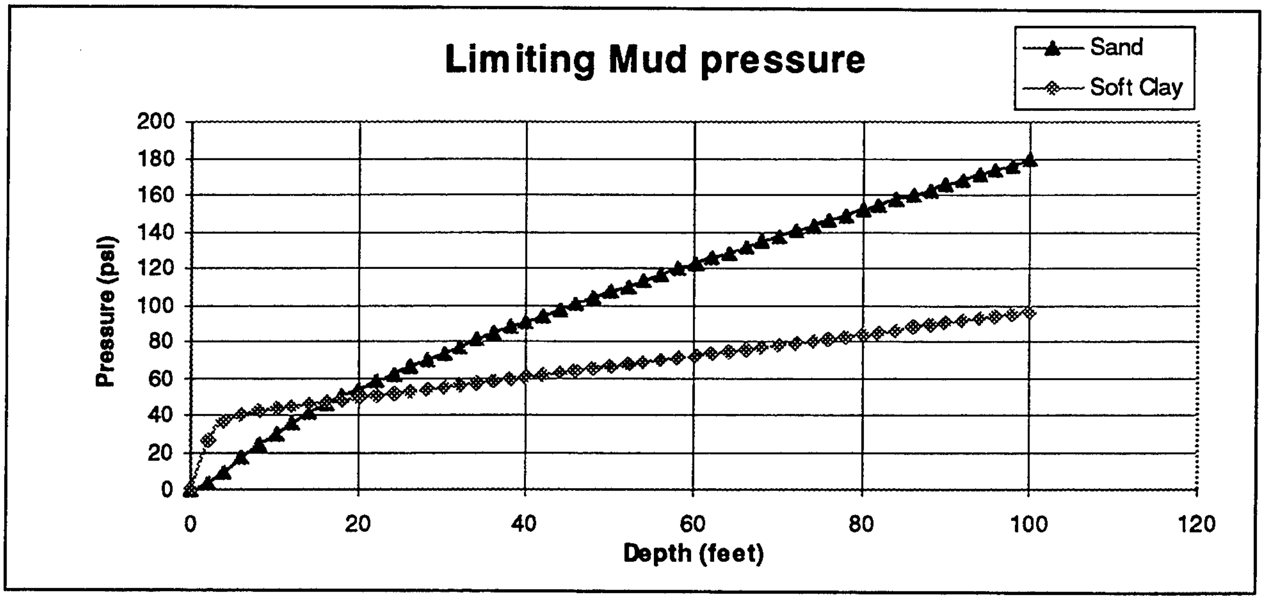


Figure A1. Limiting mud pressure, sand and soft clay. (To convert feet to meters, multiply by 0.305; to convert psi to kN/m^2, multiply by 6.89)

Table A1
Soil Properties Used for Calculating Limiting Drilling Fluid Pressures

Soil Type	Friction Angle radians (degrees)	Shear Modulus kg/m^2 (ksf)	Cohesion kN/m^2 (psf)	Unit Weight kg/m^3 (pcf)
Sand	0.51 (30)	488,431 (100)	0	1,921 (120)
Soft clay	0	122,108 (25)	2,442 (500)	1,601 (100)

Although the maximum allowable or limiting mud pressures were exceeded on the entry side of the levee, no inadvertent returns were identified because the excess pressures were dissipated through the borehole at the entry location. On the riverside of the levee, inadvertent returns were observed within 12.2 m (40 ft) of the exit location. From Figure A2 it can be seen that the inadvertent returns occurred in a zone where the drilling pressure in the annular space, which remained fairly constant at a pressure of 344.5 kN/m^2 (50 psi), exceeded the maximum allowable mud pressure.

Minimum required mud pressures

Although it is important to establish an upper bound to the pressure, it is equally important to understand that unreasonably low borehole pressures cannot be maintained without severely hindering the drilling process and, in some

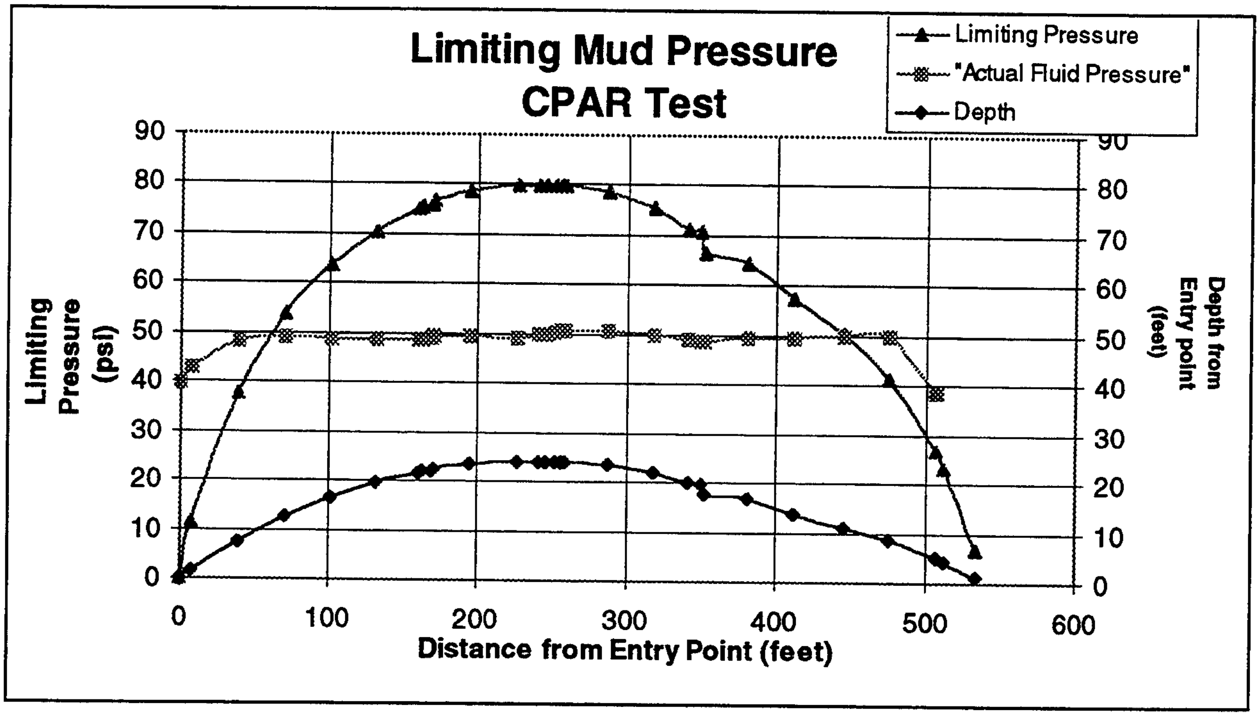


Figure A2. Limiting mud pressure, CPAR test. (To convert feet to meters, multiply by 0.305; to convert psi to kN/m^2, multiply by 6.89)

cases, making the pipe installation impossible. The drilling mud pressure must be maintained above the groundwater pressure to prevent collapse of the borehole. The pressure in the bore due to the weight of the drilling mud is calculated with Equation A2.

$$P_1 = h * \gamma_{mud} \tag{A2}$$

where

P_1 = component of minimum required annular pressure provided by mud weight

h = difference in elevation between the bore and the exit point of the mud flow

γ_{mud} = unit weight of mud

An additional component of the minimum required mud pressure is that required to start the flow of the mud with the cuttings in the bore. This component is relatively small and can be considered a threshold pressure since it is only required to start flow, not to maintain the flow of the drilling mud and the cuttings. Therefore, for simplicity, the minimum required mud pressure can be estimated with Equation A2. For the CPAR project, the average mud unit weight was 11.5 kN/m^3 (73 lb/ft^3). Figure A3 shows the minimum and maximum allowable or limiting pressures along the length of the bore.

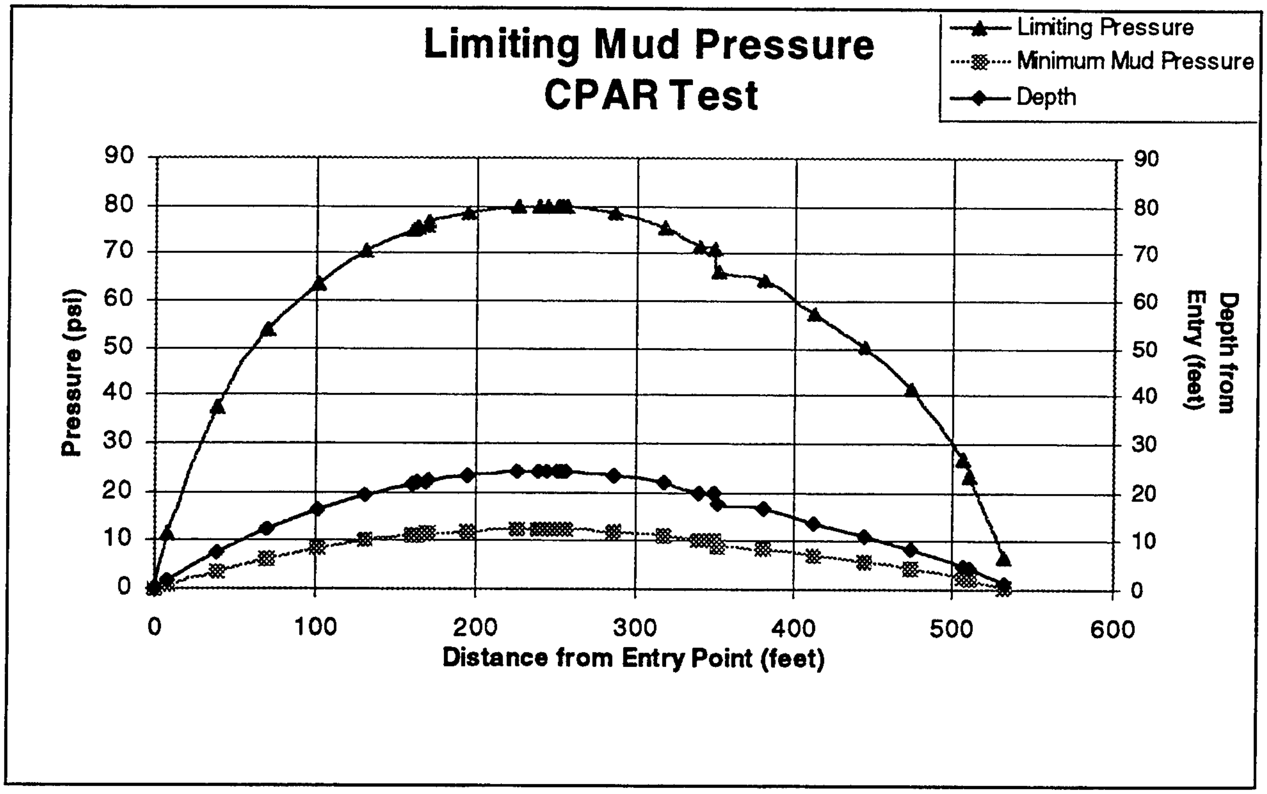


Figure A3. Limiting mud pressure, CPAR test, minimum and maximum pressure. (To convert feet to meters, multiply by 0.305; to convert psi to kN/m^2, multiply by 6.89)

During the drilling process, the required minimum pressure will vary with the groundwater head and overburden pressure. In addition, the threshold pressure required to get the fluid moving in the borehole will vary with mud weight.

Monitoring Drilling Fluid Pressures

During the drilling process, the pressure in the borehole must be monitored to ensure that the operational drilling pressures remain within the safe limits, as calculated with the recommended methods above. It is common practice to have a pressure gauge located at the mud pump to measure mud pressures within the drilling stem. However, there is a significant amount of head loss due to the flow through the drill stem and the rotational movement of the drilling mud caused by the abrupt change in flow direction as it exits the drilling stem into the annular space. The most common method for establishing the limiting pressure is to estimate the head loss and control the operational pressures, as measured at the pump, based on this estimated. However, the actual head loss is very difficult to quantify, and estimates on the head loss may lead to the establishment of limiting pressures that are not consistent with the actual conditions.

Instead of monitoring the pressure in the drill stem and estimating the head losses through the drill stem and nozzles, it is highly recommended to monitor the

pressure in the annular space, since the pressure in the borehole ultimately affects the stability of the bore. It is recommended that an external pressure measuring device, such as the device used on the CPAR project, be required on all projects when drilling beneath flood control structures. Readings provided by a down-hole pressure sensor can be used to monitor the limiting drilling pressures and ensure that the maximum allowable pressure is not exceeded. In addition, pressures should be monitored and recorded at the drill stem and in nearby piezometers, as on the CPAR project, to monitor the radial effect of the drilling process. Monitoring should include preconstruction and postconstruction readings of piezometers to establish a baseline pressure and ensure that any excess pressures resulting from the drilling process dissipate. The contractor should be required to submit plans for monitoring and controlling drilling fluid pressures and for avoiding inadvertent returns. The limiting pressures should be estimated prior to construction and clearly stated in the contract documents or in the contractor submittals. The submittal requirements should include daily logs of pressure measurements and locations at frequent intervals.

Setback Distances

Determination of appropriate setback distances can be very important with respect to damage of the levee toe and seepage and uplift pressures at the point where the top stratum is penetrated by the drill string. Levee toe stability is not the controlling factor under normal circumstances but should be checked in the design as a precaution. However, seepage is a significant concern and must be addressed on a case-by-case basis as seepage is highly dependent on levee geometry, high water level, the material of the top stratum, and the material in the substratum.

Examples of the current U.S. Army Corps of Engineers District Regulations are summarized below with respect to setback distances:

a. *Case 1*. If construction plans and specifications are not supported by borings made at the project site, the pipeline must be at its maximum depth at least 91.4 m (300 ft) landside from the center line of the levee on the landside.

b. *Case 2*. If plans are supported by borings at the project site, the drill rig must penetrate the substratum at least 91.4 m (300 ft) from the levee center line on the landside and must not exit the substratum or penetrate the top stratum any closer than 91.4 m (300 ft) riverside of the levee center line (U.S. Army Engineer District, Vicksburg 1993).

The original field memorandum, written in 1988 after the Atchafalaya Project (Wells and Kemp 1981), recommended a setback distance of 91.4 m (300 ft). This document apparently established the baseline for the regulations established by the U.S. Army Engineer Districts, Vicksburg and New Orleans. However, these restrictions were established on the observations of one project where suspect drilling conditions and procedures led to significant problems. These unfavorable conditions and procedures should be avoided, and the problems

observed on this project may not be prudent concerns in all cases. It is more reasonable to establish set back distances based on rational seepage analyses, using measured soil properties and engineering characteristics determined from prudent geotechnical investigations.

Levee toe stability

The tests conducted by WES and those conducted by Delft Geotechnics (Luger and Hergarden 1988) clearly showed that external drilling pressures do not pose a serious concern for levee stability if the pipeline is designed at an appropriate depth, proper drilling procedures are employed, and drilling pressures are monitored accordingly. When designing the depth of the pipeline, it is important to consider that the drilling fluid pressures may well exceed the maximum allowable drilling fluid pressure near the entry and exit locations due to the shallow depths, resulting in limited inadvertent returns. Because reasonable fluid pressures must be maintained to initiate and complete the bore, "excessive" pressures are necessary in these shallow zones. Therefore, the entry and exit locations should be located such that these zones do not threaten the safety of the levee.

Penetration of the top stratum

To address seepage and uplift concerns, it is critical to consider each levee crossing on a case-by-case basis because the seepage is highly dependent on soil properties and geometry. A parametric study was performed using the LEVEEMSU programs (Gabr, Taylor, Brizendine, and Wolff 1995) to establish a basis for approximate setback distances. The hydraulic gradient at the toe was recorded, as was the distance where the hydraulic gradient approached zero, signifying no concerns for seepage or uplift. The results were highly dependent on the difference between permeability of the top stratum and the substratum on the landside of the levee. As the permeability of the top stratum approached the permeability of the pervious substratum, the location where the hydraulic gradient approached zero became closer to the toe. This is because the excess pore pressure can be dissipated through the pervious top stratum instead of "transferring" the pressure to a location where dissipation is possible (farther from the levee toe). Although use of a low permeability blanket increases the distance from the toe at which the gradient approaches zero. Consequently, the maximum allowable gradient criterion should not be used alone to establish setback distances.

The LEVEEMSU program was used to analyze levee underseepage and to define reasonable setback distances. LEVEEMSU analysis algorithms are based on a numerical analysis of the flow domain and geometric conditions. The solution algorithm was based on the use of a finite difference formulation to model the steady-state flow domain. In this analysis, a two-layer model was created, with seepage flow assumed to be horizontal in the substratum and vertical in the top blanket. Hydraulic heads and gradients were computed as a function of horizontal location.

For this parametric study, LEVEEMSU calculated the hydraulic gradient at the levee toe and the horizontal distances from the levee toe to where the hydraulic

gradient was equal to 0.6 and where it was effectively zero. The layer permeabilities and the geometric properties varied over a series of 12 computations. In the first eight computations, a 15.3-m- (50-ft-) thick substratum and a 0.6-m- (2-ft-) thick top blanket was used. The water level was 17 ft (5.2 m) above the top blanket. The eight runs were produced by combining four substratum permeability values, ranging from 4×10^{-5} to 4×10^{-2} cm/sec, with two top-blanket permeability values, 1×10^{-5} cm/sec and 1×10^{-4} cm/sec. For the final four computations, the same four substratum permeability values were used with a top-blanket permeability of 1×10^{-5} cm/sec. The thickness of the substratum was changed to 14.3 m (47 ft), and the top blanket was increased to 1.5 m (5 ft) in thickness. The water level was kept at 5.2 m (17 ft) above the top blanket.

The results of this small study show that the layer thickness is not a critical factor in the resulting hydraulic gradient; however, the permeability values are significant. For all three sets of four computations, the same trend is observed: as substratum permeability decreases, the hydraulic gradient at the toe and the two recorded distances also decreases. Since this analysis involves the steady-state flow domain, it is not only the actual permeability values that account for this trend, but it is also the difference in order of magnitude between the top blanket and substratum permeabilities. When comparing different runs which have the same difference in order of magnitude for the top stratum and substratum permeabilities, similar or identical distances were calculated. Table A2 details the results from the parametric study.

Table A2
Results of Parametric Study

Top-Blanket Vertical Permeability	Substratum Horizontal Permeability	Top-Blanket Thickness m (ft)	Hydraulic Gradient at Levee Toe	Location where Hydraulic Gradient = 0.6 m (ft)	Location where Hydraulic Gradient = 0 m (ft)
1×10^{-5}	4×10^{-2}	0.6 (2)	6	305 (1,000)	549 (1,800)
1×10^{-5}	4×10^{-3}	0.6 (2)	3.8	106.8 (350)	244 (800)
1×10^{-5}	4×10^{-4}	0.6 (2)	1.1	7.6 (25)	21.4 (70)
1×10^{-5}	4×10^{-5}	0.6 (2)	0.3	--	7.6 (25)
1×10^{-4}	4×10^{-2}	0.6 (2)	3.8	106.8 (350)	244 (800)
1×10^{-4}	4×10^{-3}	0.6 (2)	1.9	22.9 (75)	61 (200)
1×10^{-4}	4×10^{-4}	0.6 (2)	0.7	7.6 (25)	15.3 (50)
1×10^{-4}	4×10^{-5}	0.6 (2)	0.1	--	7.6 (25)
1×10^{-5}	4×10^{-2}	1.5 (5)	2.5	305 (1,000)	549 (1,800)
1×10^{-5}	4×10^{-3}	1.5 (5)	1.8	106.8 (350)	244 (800)
1×10^{-5}	4×10^{-4}	1.5 (5)	1.0	15.3 (50)	68.6 (225)
1×10^{-5}	4×10^{-5}	1.5 (5)	0.4	--	15.3 (50)

The results of the parametric study clearly slow that the permeability of the substratum and top blanket are of critical importance when establishing a minimum setback distance. For projects with site conditions like the CPAR project, where the top and bottom strata were very similar materials with relatively high permeabilities, the computed minimum setback distance may be very low. This condition results, as noted previously, because the excessive pressures and high

gradients dissipate rapidly when there is little contrast in hydraulic conductivity between the top and substrata. However, if a larger contrast exists between top stratum and substratum permeabilities, the computed setback distances may be quite high. At a minimum, it is recommended that the pipeline should not penetrate any berm of the levee on either side. In cases where the difference in permeabilities between the top and bottom strata are several orders of magnitude apart, it is important to establish a reasonable distance where seepage and uplift pressures will have negligible effect son levee stability. For example, if the seepage calculations show that a setback distance of 549 m (1,800 ft) is required, one must consider if seepage 152.4 m (500 ft) (or less) from the levee would be of any concern to safety and performance of the levee.

Depth of Cover

The minimum depth of cover should be established by the calculations for maximum borehole pressures and a comparison of those pressures and reasonable drilling pressures. In the case where the reasonable operational drilling pressure exceeds the maximum drilling pressure, the pipeline should be set at a deeper elevation to raise the maximum drilling pressure. Establishing a minimum setback distance at which the maximum depth of the bore is reached prior to the center line of the levee should not be necessary as long as drilling pressures are closely monitored and remain within the established limiting pressures.

Speed of Drilling

The speed of drilling (rate at which the pipe string or pipeline is advanced through the ground) should be controlled for several reasons. It may be difficult to maintain the planned line and grade if the advance rate is extremely high. If the drill veers off line due to the advance rate, the driller may decide to pull back a section and redrill for position. The U.S. Army Engineer Waterways Experiment Station (WES) CPAR tests clearly showed that redrilling caused localized pressure bulbs that resulted in increased drilling pressures over longer time periods compared to one-pass drilling. Redrilling for position may be necessary; however, it is recommended advance rates be limited as a preventative measure against pressure buildup. It is extremely important to adjust the flow rate of the drilling mud when changing the speed of the drilling operation. This will limit the possibility of over pressurizing the borehole due to the total volume of mud that is pumped per drill pipe section.

Groundwater

The results of the CPAR tests indicated that the presence of groundwater decreased the potential for inadvertent returns to the surface, as no fracturing was observed below the water table on the project. This is due to the fact that the

pipeline was constructed primarily in noncohesive soils. Noncohesive soils do not exhibit tensile strength. As a result, tension cracks cannot propagate through the soil mass. In addition, the groundwater pressures tend to counterbalance drilling fluid pressures and reduce the potential for hydrofracture. The beneficial effect may not be realized in clay soils, because they may exhibit tensile strength in the saturated or partially saturated state. However, even in clay soils, the presence of groundwater will serve to heal old desiccation cracks that would provide a potential flow path for the pressurized drilling fluid. When practical, it is recommended that the design depth of the pipeline should remain below the water table when drilling within a lateral distance of 7.6 m (25 ft) of the levee toe.

Prevention of Seepage and Erosion Along Pipeline

The directional drilling process creates a borehole that is approximately 0.305 m (1 ft) larger in diameter than the installed pipeline. The oversized borehole is necessary to allow the pipeline to be pulled back from the exit side of the crossing without exceeding the tensile strength of the pipe and drilling stem or the pullback capacity of the drill rig. The borehole is kept filled with a fluid mixture of bentonite, water, and excavated soil during the entire process of pilot hole drilling, reaming, and pullback. The drilling fluid-soil mixture, which is comprised partially of sodium montmorillonite clay mineral, has a very low coefficient of permeability.

Concerns have been expressed about the potential for development of preferential seepage pathways along the pipeline annulus during flooding or high water stages. It has been suggested that the high hydrostatic head and gradients could cause the drilling fluid and soil mixture to be flushed from the annular space. Seepage flows around the pipeline could produce high seepage velocities resulting in soil erosion and development of boils on the landside at the point where the HDD-installed pipeline penetrated the ground. Worst case scenario would be failure of the levee system and catastrophic flooding. Depending on the drilling mud-soil mixture around the pipeline, it may not be possible to displace the material in the annulus; however, these concerns can be addressed in design and construction. The recommendations presented below focus on the design and construction measures that have been suggested by various individuals to minimize or eliminate the potential for unacceptable seepage along the pipeline. These measures include:

a. Grouting of annular space and minimizing annular space.

b. Landside seepage blankets or berms.

c. Riverside cutoffs or collars (applicable only for pipelines that exit or enter the riverside of levee).

Grouting of annular space

Grouting of the annular space with a cement or bentonite-cement grout mixture has been suggested or required on some pipeline crossings. The objective has been to expel the semifluid mixture of bentonite, soil, and water with a grout material that will set and provide a solid barrier against seepage flow along the annulus. One possibility is that a grout mixture with a delayed set time be pumped into the hole during the final reaming and pullback of the pipe to more effectively displace the bentonite based drilling mud mixture. It is argued that this process would reliably and completely expel the drilling fluid and replace it with grout.

The proposal for grouting during pullback reduces the risks of future development of seepage pathways. However, the risks of failure to complete the pipeline installation could be high. If for any reason the pullback was delayed beyond the initial set time, the partially installed pipeline could become grouted in place. Substantial financial loss would be incurred by the pipeline company and/or contractor. In addition, the problem of a partially installed pipeline would have to somehow be mitigated.

The field research performed under the CPAR program could not address this issue. While filling the annular space with a low-permeability material is a desirable goal, the process of grouting during pullback is not recommended. Research and testing of grout materials with controlled delayed set times and grouting procedures should be required prior to such a recommendation. At this point, the potential risks of failure to complete the installation outweigh the perceived benefits of more reliably filling the borehole with a bentonite-cement grout. The risks of failure would impact the Corps of Engineers, Levee Boards, and the general public, as well as the contractors and pipeline operating companies.

Grouting of the annular space upon completion of the bore should also be addressed. The grouting pressures required to expel the drilling fluid must exceed hydrostatic pressures because the drilling fluid pressure in the annulus must equal or exceed hydrostatic pressure. The grouting pressures must be lower than the overburden pressure or critical pressure required to initiate hydraulic fracturing. To increase the likelihood of uniform grout distribution around the pipe annulus, the use of perforated grout tubes attached to the pipeline has been suggested. After the grout is pumped through the tubes, they would be abandoned in place. This process would increase the difficulty and risk of failure of the pullback operation and could adversely impact corrosion resistance of the pipeline. This procedure was not tested as part of the CPAR field evaluation. Additional research to help establish the reliability of this grouting procedure may be beneficial.

A grouting procedure that may be viewed as a compromise may hold promise and is recommended. In this procedure, grouting tubes would be inserted as far as possible into the borehole after the pipe is pulled back. The grout mixture would be pumped into the annulus through these tubes until grout returned to the surface at the entry or exit of the pipeline. Grouting pressures must be carefully controlled to minimize risks of hydrofracture. This process may not be completely effective in dispelling drilling fluid and providing a low-permeability, solid barrier to seepage. However, the results should be beneficial if carried out carefully.

This procedure is recommended as an added insurance measure at both ends of the pipeline.

In addition, the composition and hydraulic conductivity of the soil-drilling fluid mixture should be tested prior to construction to determine the in-place resistance to seepage provided by the mixture. It may be determined that the hydraulic conductivity of the soil-bentonite-water mixture is sufficiently low (lower than the surrounding natural soil) to minimize potential for seepage along this pathway. These tests should be performed using the actual drilling fluid mixture(s) planned for use on the project, with varying percentages of bentonite and natural soils to bracket the planned or expected field conditions. This approach would also necessitate field quality control tests to ensure that the drilling fluid mixtures used for construction were the same as those tested.

Seepage blankets or berms (antiseepage devices)

Seepage blankets and berms have been used for many years to increase the factor of safety against piping and erosion along the landside toe of levees. Design of seepage blankets and berms is covered in EM 1110-2-1913 (Headquarters, U.S. Army Corps of Engineers (HQUSACE) 1978). Some form of these features could be used on the landside entry and exit points of pipeline crossings for the same purpose, i.e., to reduce the risk of piping and erosion along the pipeline that could undermine the levee or its foundation. However, the specific criteria in EM 1110-2-1913 calls for installation of a drainage fill with an annular thickness of 0.457 m (18 in.) around the landside third of the pipe. This is not feasible with HDD pipeline installation.

Instead, it is recommended that a seepage analysis be performed during design of the crossing. If the hydraulic gradient at the landside entry/exit points exceeds the maximum allowable gradient, a landside seepage blanket should be evaluated. If the provision of the seepage blanket increases the factor of safety against piping to an acceptable level, it may be an economical insurance feature. To achieve its design function, the blanket would not have to extend great distances on either side to the pipeline, but could rather be a small localized surface feature with gentle slopes to aid in levee maintenance. Depending on design requirements, the seepage blanket might add only very small cost to the project, yet provide significant benefits. The evaluation should be performed using actual soil properties, site conditions, and geometry.

Riverside cutoffs or collars

Riverside cutoffs or seepage collars may be considered for projects with exit points on the riverside of levees. For projects that enter and exit on the landside of opposite bank levees, riverside cutoffs are obviously not applicable. Seepage barriers, rings, or cutoffs are addressed in EM 1110-2-1913 (HQUSACE 1978). The benefits of and need for seepage barriers or collars have been questioned. Poor compaction has been cited in EM 1110-2-1913 as a cause of piping failures with these devices, and their use is discouraged in the manual. If considered, seepage collars should be evaluated during design using actual site conditions, soil

properties, and geometry. However, quality control must be meticulous to ensure that design objectives will be met. Specifically, the materials, mixture, placement, and compaction are all critical design elements. The materials and mixture should ensure low hydraulic conductivity, low shrinkage, and long-term stability. Placement and compaction must ensure intimate contact around the full pipeline circumference, without damage to the pipe. Laboratory tests of the hydraulic conductivity of the materials and mixture should be required. In addition, hydraulic conductivity tests of the system may be beneficial. This could be accomplished in the lab using a small-scale model of the system, i.e., the mixture placed and compacted to design specifications around a tube to simulate the pipeline. The collar must extend for a sufficient distance around the pipeline to provide an effective impediment to seepage. Dimensions can be established by sequential seepage analyses with different trial dimensions.

Closure Devices

Closure devices are required in EM 1110-2-1913 (HQUSACE 1978) for all pipes that penetrate the embankment or foundation of a levee. Flap valves or gate valves are recommended and automatic devices are described with design guidance provided in EM 1110-2-1410 (HQUSACE 1965). Closure devices (valves) could serve a critical purpose in an emergency and should be considered with regard to pipelines beneath levees. Values are required for liquefied petroleum pipelines by U.S. Department of Transportation regulation, Part 195, Section 260(e), at water crossings longer than 30.48 m (100 ft). Valves are not required on gas pipelines since there is no danger of spills.

Relief Wells

Relief wells have been proposed and used on a number of projects involving HDD; however, relief wells are not considered necessary under normal circumstances. The objective of proposed relief wells has been to vent the high drilling fluid injection pressures and avoid fluid pressures that exceed earth and groundwater pressures. The directional drilling process uses relatively high drilling fluid pressures and flow rates to the injection nozzle. These reported pressures have caused concerns about hydrofracturing. However, it should be understood that these pressures are quickly attenuated within a short distance of the nozzle. In the tests conducted by WES, and in those conducted by Delft Geotechnics (Luger and Hergarden 1988), the pressures measured in the annular space between the pipe or drill stem and the borehole wall were significantly lower than the nozzle pressures. In the WES tests, pressures in the annular space were only 323.83 to 358.28 kN/m^2 (47 to 52 psi), even for internal pressures as high as 2,411.5 kN/m^2 (350 psi). Excess pore pressures as recorded by the piezometers were less than or equal to 6.89 kN/m^2 (1 psi), and these excess pore pressures dissipated rapidly. Based on these results, relief wells are not considered necessary for venting drilling fluid pressures. Relief wells may be effective for dissipating high seepage pressures on the landside toe of levees during high water events. This application is well documented and different from their use for venting drilling fluid pressures.

References

American Gas Association, Pipeline Research Committee. (1994). "Drilling fluids in pipeline installation by horizontal directional drilling." *Practical application manual*. PR-227-9321, Tulsa, OK.

Brizendine, A. L., Taylor, H. M., and Gabr, M. A. (1995). "LEVSEEP: Analysis software for levee underseepage and rehabilitation," Technical Report GL-95-10, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Delft Geotechnics. (1997). A report by Department of Foundations and Underground Engineering prepared for O'Donnell Associates, Inc., Sugarland, TX.

Gabr, M. A., Taylor, Hugh M., Brizendine, Anthony L., and Wolff, Thomas. (1995). "LEVEEMSU: Analysis software for levee underseepage and rehabilitation," Technical Report GL-95-9, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Hair, J. D., and Associates, Inc., Cappozoli, Louis J., and Associates, Inc., and Stress Engineering Services, Inc. (1995). "Installation of pipelines by horizontal directional drilling." *Engineering design guide*. PR-227-9424, Tulsa, OK.

Headquarters, U.S. Army Corps of Engineers. (1965). "Interior drainage of leveed urban areas: Hydrology," Engineer Manual EM 1110-2-1410, Washington, DC.

__________. (1978). "Design and construction of levees," Engineer Manual 1110-2-1913, Washington, DC.

Luger, H. J., and Hergarden, H. J. A. M. (1988). "Directional drilling in soft soils: Influence of mud pressures." *Proceedings, International No-Dig '88*. Washington, DC.

U.S. Army Engineer District, Vicksburg. (1993). "Project operations—Project maintenance standards and procedures," District Regulations 1130-2-303, Vicksburg, MS.

Wells, J. T., and Kemp, G. P. (1981). "Atchafalaya mudstream and recent mudclat progradation; Louisiana chenier plain," *31st annual meeting, Gulf Coast Association of Geological Societies and 25th annual meeting of Gulf Coast Section of Society of Economic Paleontologists and Mineralogists*. Corpus Christi, TX.

Wolff, T. F. (1989). "LEVEEMSU: A software package designed for levee underseepage analysis," Technical Report GL-89-13, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Appendix B
Delft Geotechnics Report

Research (CPAR) program
Installation of Pipelines Beneath Levees
Using Horizontal Directional Drilling

SO-59407-701/2
September 1997

The report was prepared for:
O'Donnell Associates, Inc.
P.O. Box 1208
Sugarland, TX 77487, USA

FOUNDATIONS AND UNDERGROUND ENGINEERING DEPARTMENT
project manager: G. G. van Brussel
project supervisor: H. J. A. M. Hergarden

DELFT GEOTECHNICS
Stieltjesweg 2, 2628 CK DELFT
P.O. Box 69, 2600 AB DELFT

Telephone 31 - 15 - 269 35 00
Telefax 31 - 15 - 261 08 21
Postal account 234342
Bank MeesPierson NV
Account 25.92.35.911

Report no.: SO-59407-701/2	Date report: 1977-09-02
Title and subtitle: Construction Productivity Advancement Research (CPAR) program Installation of Pipelines Beneath Levees Using Horizontal Directional Drilling	Department Foundations and Underground Engineering Project:
Project manager(s): G. G. van Brussel	Project supervisor(s): H. J. A. M. Hergarden
Name and address of client: O'Donnell Associates, Inc. P.O. Box 1208 Sugarland, TX 77487, USA	Reference client: CPAR-GL-97-5 Copies sent: 3 Type report: final

Summary of report:

Delft Geotechnics has offered support to the CPAR program: Horizontal Directional Drilling. The CPAR program is carried out by the Government and the industry to investigate how to install pipelines under the leveed banks of the lower Mississippi River by use of the Horizontal Directional Drilling techniques. The case of bore 1 is analyzed.

This report contains 3 sections which address the following subjects:

1. Interpretation of soil investigation results.
2. Calculation of the maximum allowable mud pressure during the pilot bore and the ream and pullback operation of the pipeline.
3. Calculation of the minimum required and mud pressure during the pilot bore and the ream and pullback operation of the pipeline.

Further, a discussion about the CPAR report is presented over the calculation result of the maximum allowable mud pressure. In Chapter 7 of this report by Delft Geotechnics, parts from the Dutch guidelines are presented for a save installation of pipelines by using the Horizontal Directional Drilling (HDD) method.

Comments:

Keyword: directional drilling	Distribution: O'Donnell Associates, Inc.
Saved under title:	No. of pages:

Version:	Date:	Prepared by:	Signature:	Checked by:	Signature:
01	1997-09-02	Brus		Lug/Her	
02	1997-09-19	Brus		Her	

Delft Geotechnics

1 Introduction

Delft Geotechnics has offered support to the CPAR research program: Horizontal Directional Drilling. The CPAR research program is carried out by the Government and the industry. The objective of the research is to develop guidelines for installing pipelines under the leveed banks of the lower Mississippi River by use of the Horizontal Directional Drilling Techniques. The support of the program by Delft Geotechnics will consist of a review of the report and comments on the results.

In this report, the case of bore 1 is analyzed. The following subjects will be discussed:

a. Interpretation of soil investigation results.

b. Calculation of the maximum allowable mud pressure during the pilot bore, the ream, and pullback operation of the pipeline.

c. Calculation of the minimum required mud pressure during the pilot bore, the ream, and pullback operation of the pipeline.

For information, parts of the Dutch guidelines are presented in Chapter 7.

2 Project Description

The levee that is crossed by the tests bores was an old landside setback portion of the mainline Mississippi River Levee, built in 1941 during a period of rapid channel migration. The geology of the site and area has been shaped by the Mississippi River. This site represents a good sample for the rest of encountered crossings.

Three test bores were planned beneath the earthfill levee located in Mayersville, MS. The length of each boring was about 161 m (530 ft). The entry point of the three bores are at the river side of the levee. The entry angle is about 0.21 radians (11.5 deg). The designed radius of the bore is the outer diameter of the pipeline (366 m) multiplied by 1,200. The maximum depth under the dike is about 14 m (46.5 ft). The angle of the bore at the exit point, located near Carlisle Lake, is about 0.14 radians (8 deg).

For the outer diameter of the pilot string of the first bore, a 114-m (5-in.) pipe was used. The diameter of the created borehole by the "tri-cone" is about 222 mm (8-3/4 in). Upon completion of the pilot bore, a 510-mm (20-in.) fly cutter was attached to the drill sting, along with a 305-mm- (12-in.-) diam steel pipe string. The hole was then reamed and the product pipe installed in one pass.

3 Interpretation of Soil Investigation Results

Based on the results of this report, CPAR-GL-98-1, the soil profile at the proposed route (bore 1) is as follows:

Number of layer:

a. From ground surface level (about ELEV +0.27 to +6.55 m) to approximately ELEV -0.3 to +5.5 m: a medium to very stiff tan to gray silty clay. The level of the dike is about ELEV +6.55 m. The level of the entry point of the Horizontal Directional Drilling (HDD) is about ELEV +0.27 m and the level of the exit point is about ELEV +0.91 m.

b. The body of the dike from about ELEV +5.5 m to approximately ELEV +0.5 m: a layer of sand consisting of clay and silt.

c. From ELEV -0.3 to +0.5 m to approximately ELEV -3.3 m to ELEV -5.5 m: a layer with fine, tan and gray, clayey and silty fine sands of loose to dense consistency.

d. From ELEV -3.3 m to ELEV -5.5 m to approximately ELEV -13.0 m: a layer of silty fine sands.

The phreatic groundwater level fluctuates with river levels. The groundwater level is about ELEV -0.25 m.

For the calculation of the maximum allowable mud pressure, the following soil characteristics have been used:

Layer 1:	Total unit weight	$\gamma_{\text{above groundwater level}} = 14.0$ kN/m^3 and $\gamma_{wet} = 16.0$ kN/m^3
	Internal friction angle	$\phi = 20.0°$
	Cohesion	$c = 5$ kN/m^2
	Undrained shear strength	$c_u = 25$ kN/m^2
	Shear modulus	$G = 714$ kN/m^2

Layer 2:	Total unit weight	$\gamma_{wet} = 18$ kN/m^3
	Internal friction angle	$\phi = 27.5°$
	Cohesion	c = 2 kN/m^2
	Shear modulus	G = 3,703 kN/m^2
Layer 3:	Total unit weight	$\gamma_{wet} = 19$ kN/m^3
	Internal friction angle	$\phi = 30.0°$
	Cohesion	c = 0 kN/m^2
	Shear modulus	G = 5,555 kN/m^2
Layer 4:	Total unit weight	$\gamma_{wet} = 20$ kN/m^3
	Internal friction angle	$\phi = 32.5^0$
	Cohesion	c = 0 kN/m^2
	Shear modulus	G = 9,615 kN/m^2

4 Maximum Allowable Mud Pressure

During the various stages of the directional drilling process, drill mud is injected near the drilling front. This drill mud has to serve several purposes:

a. Jetting of the soil.

b. Removal and transportation of the cuttings.

c. Stabilization of the borehole.

d. Lubrication for the pulling of the pipeline.

These functions require a certain drill mud pressure which, however, has to be kept within certain bounds to prevent borehole collapse at low pressures or uncontrolled expansion or hydraulic fracturing at high pressures.

The maximum allowable mud pressure is calculated using the "Delft Geotechnics"[1] method which is based on the cavity expansion theory.

The equation used for the calculation of maximum allowable mud pressure in the borehole is as follows:

$$p_{\max} = u + p'_{\max} = u + (p'_f + c \,.\, \cot\phi) . \left\{ \left(\frac{R_0}{R_{p,\max}} \right)^2 + Q \right\}^{\frac{-\sin\phi}{1+\sin\phi}} - c \,.\, \cot\phi$$

where

$p'_{\max}$ = maximum allowable effective mud pressure in N/mm^2

p'_f = mud pressure at which the first plastic deformation takes place in N/mm^2, $p'_f = \sigma'_0 .(1+\sin\phi)+c.\cos\phi$

σ'_0 = initial effective stress in N/mm^2

---

1 Delft Geotechnics. (1990). A report by Department of Foundations and Underground Engineering prepared for O'Donnell Associates, Inc., Sugerland, TX.

ϕ = angle of internal friction in degrees

c = cohesion in N/mm^2 (in undrained situation $c = c_u$ and $\phi = 0$)

R_0 = initial radius of the bore hole in mm

$Q = \sigma'_0.\sin\phi + c.\cos\phi/G$

G = shear modulus in N/mm^2

$R_{p,\max}$ = radius of the plastic zone

u = initial pore pressure in N/mm^2

To prevent blowouts, the plastic zone has to remain within a safe radius around the borehole. In clayey and peat layers, the maximum allowable radius of the plastic zone is chosen as ($R_{p,\max}$) = H/2 and in sand layers $R_{p,\max}$ = 2/3 H, where H = height of the soil cover.

The data of Table B1 are used in the calculations for the mud pressure.

Table B1
Input Data for Calculation of Maximum Allowable Mud Pressure

Parameters	Pilot Bore	Ream and Pullback Operation
Outer diameter pipe	114 mm (5 in.)	305 mm (12 in.)
Outer diameter borehole	222 mm (8-3/4 in.)	510 mm (20 in.)

The result of the calculations of the maximum allowable mud pressure ($P_{\max}$) during the various phases are presented in a longitudinal profile.

5 Minimum Required Mud Pressure

Beyond the maximum allowable mud pressure, also the minimum mud pressure for a sufficient mud flow is calculated.

To carry out horizontal directional drilling, a certain minimum pressure is needed in the borehole. This minimum pressure is required to transport the cuttings out of the borehole. When the minimum required mud pressure exceeds the maximum allowable value, the pipeline has to be installed at a lower level.

In the borehole, a certain minimum mud pressure (P_{min}) is required. This minimum required pressure at the bore front depends on two aspects:

a. The difference in height between the bore front and the exit point of the mud flow to the surface.

b. The minimum pressure that is required to start the flow of the mud with the cuttings in the bore hole over a certain distance.

Seeing that the bore front is lower than the exit point of the mud flow, a pressure difference must be overcome that is approximately equal to the difference in height multiplied by the density of the mud. This is given in the following formula:

$$P_1 = h * \gamma$$

where

P_1 = contribution to minimum required mud pressure [kPa]

h = difference in height between level of bore front - exit point of the mud flow [m]

γ = unit weight mud [kN/m^3]

To create a flow of the mud in the borehole, the shear resistance of the mud has to be conquered. During the operation there will be a borehole with a pilot pipe in it. Between the outside of the pipe and the wall of the bore hole (annulus), there will be a flow of the drilling fluid. The required pressure depends on the

dimension of the space between the drill pipe or product pipe and the borehole, properties of the drilling fluid, and the desired mud flow. The minimum required mud pressure can be calculated. The drilling fluid can be considered as a so-called Bingham plastic fluid with a yield point τ_0 and a plastic viscosity μ_0. This calculation leads to a value for the resistance that the flow of the fluid will experience along the pipe and the borehole (*dp/dl*). The contribution P_2 for the minimum required fluid pressure is as follows:

$$P_2 = \frac{dp}{dl}.l$$

where

dp/dl = required pressure by length unit of the borehole (kPa/m)

l = the distance along the borehole from the position of the bore front up to the exit point of the drilling fluid (m)

The total minimum required pressure is calculated as follows:

$$P_{min} = P_1 + P_2 = h*\gamma + \frac{dp}{dl}*l$$

The data of Tables B1 and B2 are used in the calculation for the minimum needed mud pressure.

Table B2
Input Data for the Calculation of the Minimum Required Mud Pressure

Parameters	Pilot Bore	Ream and Pullback Operation
Q mud flow	570 l/min (150 gpm) (00095 m^3/s)	948 l/min (250 gpm) (00158 m^3/s)
Average unit weight mud	115 kN/m^3	115 kN/m^3
τ_0 yieldpoint of mud	002 kPa	002 kPa
Plastic viscosity μ_0 of mud	0.4 * 10^{-4} kPa.s	0.4 * 10^{-4} kPa.s

6 Conclusions of the Results

The geotechnical investigation carried out at the site has indicated from top to bottom a soft layer of silty clay, a clayey and silty fine sand layer, and a silty fine sand layer. For the calculation of the mud pressure in the top layer, the undrained soil characteristics are used, and for the other layers the drained soil characteristics are used.

For the design of a pipeline crossing carried out by a directional drilling technique, it is not necessary to calculate the pressure loss between drilling rig and bore front.

During the pilot bore, the values of the minimum required mud pressure near by the exit point (starting between x-coordinate 140 and 147.5 m to x-coordinate 167 m) are higher than the maximum allowable mud pressures. It is possible that over this distance a blowout of the drilling fluid can occur. Since the distance to the exit point is not that big, the occurrence of a blowout may not be a problem.

But to overcome an early blowout, the following measures can be taken:

- Creating a bigger borehole over the total length of the pilot bore. Problems with steering the pilot string can occur when the borehole is too large.

- Reduce the flow rate of the fluid.

- Decrease the speed of drilling.

- Enlarge the angle to about 0.19 or 0.21 radians (11 or 12 deg) at the exit point. The cover near the exit point will increase and also the maximum allowable mud pressure. If the angle is too large, problems can occur constructing the pipe on the rollers for the pulling operation.

During the ream and pullback operation, the minimum required mud pressure stays below the maximum allowable mud pressure. If a bigger borehole is created during the pilot bore, the mud can flow more easily to the exit point or to the entry point when the ream and pullback operation is carried out. The needed mud pressure will then be much lower.

From the results of the calculation, we can conclude that the cover depth of the pipeline in these phases is sufficient. The influence of the boring on the levee will be negligible.

7 Guidelines for an HDD Design

In The Netherlands there are standards that embody the general requirements imposed on the design, construction, and the operation of steel pipeline transportation systems and the related aspects of safety, the environment, and public health.

The following standards are applied for making an HDD design:

a. NEN 3650--Requirements for steel pipeline transportation systems.

b. NEN 3651--Supplementary requirements for steel pipelines crossing major public works (dykes, high level canals, water ways, roads).

c. NEN 3652--Additional requirements for nonsteel pipelines in crossings of important public works.

From these standards, the following text is presented:

page 40 of NEN 3651

6.4.1 General

The entry and exit points of horizontal drilling must lie outside the dike crossing and safety zones.

(The design (depth) of the HDD must be such that during the operation the stability of the dikes or the bottom of canals/rivers are not influenced.)

6.4.2 Soil-mechanics parameters

In addition to the usual data, supplementary soil-mechanics investigations are required of such areas as:

- The structure of the deeper subgrade strata;
- Acidity measurements for the purposes of determining the drilling mud composition;

- Identification of any gravel strata or clean shell beds which might make this construction method difficult to use;
- Calculation of the maximum allowable pressure in the drilling fluid in relation to the depth of cover;
- Determination of the minimum depth at which drilling is permitted, in the interests of the stability of the public work. Instability can be caused by increased pore water pressure during drilling;
- The permeability of the different soil strata for the purpose of comparing natural seepage paths with the seepage path along the borehole;
- The pore pressure in water bearing sand layers if it is necessary to penetrate these layers.

Note: Cone-penetration tests must be conducted and boreholes sunk some distance (5-10 m) away from the projected axis of the pipeline to prevent the risk of escape of drilling fluid.

Directional horizontal drilling techniques must not be used if it is possible for a seepage path to be created along the pipeline which has a lower hydraulic resistance than the shortest 'natural' seepage path, unless the pipeline running beneath the public work (dike) and the safety zones lies entirely within a sand layer.

page 54 of NEN 3651

7.6.1 General

Since the construction of pipeline crossings by the horizontal directional drilling method minimizes or entirely eliminates any disturbance of the public work concerned, this is the generally preferred method.

With the horizontal directional drilling method, the soil can be either removed or expelled. Tests have shown that the excess pore water pressure generated in the vicinity of the drill head is of the same order of magnitude.

A substitute water-retaining structure in the form of sheet piling is not necessary in conjunction with horizontal directional drilling in view of the inherently low-risk nature of this method.

7.6.2 Construction requirements

a) Drilling fluid
Evidence must be provided that the drilling fluid (bentonite) is sufficiently stable in connection with acidity of the soil and the salinity of the groundwater. The drilling fluid must comprise a mixture of water and bentonite with various additives as necessary. It is not permissible to carry out horizontal

directional drilling for crossings of public works using water only as drilling fluid.

b) Drilling fluid pressure
The maximum pressure in the drilling fluid during insertion of the product pipe, the wash-over pipe, or pilot string may not exceed a predetermined limit derived from soil mechanics investigation.

c) Depth of cover
Based on this pressure, a minimum depth of cover must be calculated, applying the appropriate safety factor. In clay and peat, the safety factor is 2. In sand, a safety factor of 1.5 may be taken since the probability of the maximum value being attained is less than in cohesive soils. These safety factors apply whether the soil is to be expelled or removed.

As a further requirement, it is stipulated that the minimum depth of cover beneath the crown of the dyke/bottom of surface waters must not be less than 10 m. Where a vertical drainage system exists (e.g. sand piles), the pipe must be approximately 2 m below the base of the drainage system.

Appendix C
Cappozoli Report

LOUIS J. CAPOZZOLI & ASSOCIATES, INC. Geotechnical Engineers

Charles W. Hair, III, P.E.
James M. Aronstein, Jr., P.E.
Charles L. Eustis, P.E.
David P. Sauls, P.E.

Louis J. Capozzoli, ScD, P.E.
Consultant

30 June 1996

US Army Corps of Engineers
Waterways Experiment Station
Geotechnical Laboratory
3909 Halls Ferry Road
Vicksburg, Mississippi 31980-6199

Attention: Ms. Kimberly Staheli

Re: Geotechnical Support
Embankment Subdrilling Study
Mississippi River Mile 492.9 AHP
Issaquena County, Mississippi
Purchase Order No. DACW39-96-M-1337
13 May 1996
LJC&A File: 96-56

Gentlemen:

Site characterization information relative to your studying the effects of installing a pipeline beneath an earthen embankment via the horizontal directional drilling (HDD) method is provided by the following report. A geotechnical description of the site plus geotechnically based HDD design/construction particulars - stemming from analysis of both furnished/published data as well as field exploration/laboratory testing results - constitute the text. The geotechnical support's qualitative/quantitative foundation, including laboratory evaluation and field work/instrumentation installation specifics, is described by the enclosures. To facilitate the overall study's conduct, preliminary geotechnical results were transmitted as they developed.

Performed under auspices of the captioned *Purchase Order*, site characterization execution was authorized by *Requisition Request No. W81EWF-6114-9943.* The enclosed appendix A, *Geotechnical Support,* defines our effort's structure plus administrative particulars.

<u>PROJECT DESCRIPTION</u>

<u>Location</u>. Project site positioning - on the Mississippi River's left descending (eastern) bank in northwestern Issaquena County, Mississippi - is roughly $4^1/_2$ miles south-southwestward of the Mayersville community. Further site location aspects are depicted by sheet 1.

<u>Construction.</u> To study effects of HDD on earthen embankments; you will trenchlessly emplace at this site two parallel, 12 inch nominal diameter, steel pipelines beneath an instrumented section of deactivated Mississippi River flood protection levee. These installations will be sufficiently short and shallow so that post-placement excavation/retrieval of the pipes can be accomplished. Overall objective of such construction is to quantitatively and qualitatively define:

10555 AIRLINE HIGHWAY, BATON ROUGE, LOUISIANA 70816
TELEPHONE (504) 293-2460 FAX (504) 293-2463

- the site's *passive effects* on HDD, i.e. the ease/difficulty and manner of drilling conduct the in situ stratification , plus

- the site's *active responses* to HDD, i.e. the various phenomena (especially the occurrence of inadvertent drilling fluid returns to the ground surface) associated with procedure execution.

In essence, this research is intended to advance the practicality and usefulness of HDD.

Site Evaluation. The remainder of this report's text documents the nature of - and rationale behind - our provision of geotechnical support to project planning, performance, and forensic examination. Patterned after the geotechnical investigation usually applied to the site of a pipeline or telecommunications river crossing; a summary outline of our efforts is followed by discussions of:

- the project site's geological, hydrographical/topographical, and geotechnical conditions relevant to HDD;

- the role of Mississippi River potamology (alluvial behavior) in developing the site's pertinent characteristics; and then

- the geotechnical details requisite to designing, conducting and evaluating the trenchless installation.

Concluding the report are geotechnically-based assessments/recommendations relative to project implementation.

GEOTECHNICAL SUPPORT

Site evaluation supporting study execution first entailed definition of the in situ earth materials: the soils' properties pertinent to HDD as well as the stratification's origins relative to long-term Mississippi River activity/short-term works of humankind. Coincidentally, field instrumentation - i.e. piezometers for measuring groundwater pore pressure changes - was installed. Afterwards, investigation-generated geotechnical characteristics were applied to assessing/forecasting HDD's site-specific applicability and conduct particulars. Geotechnical support details - including descriptions of purpose, constituency, and execution - are presented in appendix A plus appendix D, *Field and Laboratory Analyses*. Despite this project's research orientation, salient geotechnical aspects are generally in line with numerous recent evaluations by us of crossing sites on large rivers located throughout the southcentral United States.

SITE CONDITIONS

Details highlighted by this section are depicted on sheets 1 through 3.

General. At the latitude of your project site; the navigable, channel section managed Mississippi River is positioned in the central third of its floodplain: an 80 to 100 mile wide, generally north-to-south oriented, valley. Formation of the valley was by alluvial scouring of early Tertiary (Eocene) age, highly overconsolidated, Marine (saltwater sea deposited) stiff to very stiff strength clay. Sedimentation from the Mississippi, in thicknesses sometimes exceeding 200 feet, now partially fills the valley: elevation of the present-day valley floor near the project site averages 100 feet, National Geodetic Vertical Datum (NGVD). Numerous abandoned channels, various size natural levees (course paralleling ridges sedimented from over-bank flooding), and meander scrolling (curvilinear ridges and swales denoting the "wake" of a laterally migrating river channel) dominate near-site topography. Such natural features - ranging from flat-sloped mounds and sediment-filled depressions to water-filled lakes - indicate the river's past propensity for horizontal activity: relatively slow course relocations across its valley floor floodplain. Appendix B, *Potamological Analysis*, presents alluvial activity details.

Salient example of Mississippi River alluviation is *Carlisle Lake*, the relatively shallow body of water immediately adjacent to the project site: see sheets 1 and 2. Physical characteristics - i. e. dimensions, orientation/positioning, etc. - of this feature reveal it to be a long abandoned, now mostly sediment-filled, former bendway of the Mississippi whose present-day active channel is more than a mile to the northwest. Significantly, project site location in/on an abandoned bendway reach forecasts granular silts/sands - and possibly even point bar gravels - likely constitute the stratification to be negotiated via HDD.

At present, the dual requirements of flood control and governmental mandate to maintain the Mississippi's existing alignment have driven manmade "stabilization" of the channel. Under auspices of the U.S. Army Corps of Engineers, measures for accomplishing this near your site include: onbank flood protection *levees* (crest elevation at roughly 119 feet, NGVD); channel edge *revetments* (articulated concrete mattresses); and inchannel flow deflector *jetties*. Such facilities - by reducing the river's natural tendencies for overbank flooding and lateral migration - have temporarily "fixed" the Mississippi's present-day location relative to the project site. In concert with past artificial relocation of the nearby flood protection embankment, the course stability thus induced has "produced" the object of this research: the deactivated levee slated for subdrilling - see appendix B.

<u>Surface.</u> Within the area of interest, the ground on both sides of the object embankment is relatively level and situated at about elevation 100 feet, NGVD. Riverward - i.e. to the northwest, the flood side batture (land between the river and the levee) is covered with medium to large size hardwood trees and is subject to periodic inundation. Conversely, the landward protected side surface surrounding Carlisle Lake is under cultivation.

Both faces of the levee embankment are grass covered and periodically mowed. Side slopes approximate 4H to 1V. The crest is about 10 feet wide and serves as a travel-way for light vehicles.

Sheet 2 pictorially depicts the above described conditions.

<u>Subsurface.</u> Sheet 3 graphically displays subsurface stratification encountered by our onsite exploration. Although undetected anomalies (gravel pockets, buried logs, etc.) may exist, generalized conditions are:

Stratum Nomenclature	Inclusive Elevations Feet, NGVD	Stratum Description
<u>Man-Made Levee.</u> Artificially transported/compacted earthen embankment	Embankment Crest (119 ±) to 100/90	Twenty foot nominal height, relatively steep sloped ridge composed of soft to very stiff strength clay and loose to firm sand/silt.
<u>Topstratum.</u> Recent Alluvium deposited from/in the Mississippi's present day gross channel meander belt.	From the Above Elevations and/or The Ground Surface (100) to The Exploration's Extent	<u>Bank Surface Sedimentation.</u> A 2 to 5 foot thick "crust" of medium to stiff strength clay deposited by overbank flooding prior to levee construction. Clay strength is due mainly to desiccation (surficial drying) which also produced "slickensides" - shrinkage cracks which have subsequently healed. <u>Channel Fill.</u> Loose to firm silt and sand - containing clay sublayers/pockets - plugging the abandoned river course now defined by Carlisle Lake.

The foregoing layering typifies near-surface conditions at this latitude of the Mississippi River's alluvial valley. Deeper soils, both alluvially as well as Marine deposited, are below the zone of HDD installation interest.

<u>Groundwater.</u> Immediately adjacent to Calisle Lake, the water table is denoted by that water body's level. By about 50 feet landward, free water is usually encountered within 3 to 5

feet of the ground surface except when directly beneath the levee. There; a 6 to 8 foot high, surface contour following, groundwater "mound" can be expected. In terms of watertable fluctuation: during periods of rain and/or flooding the phreatic surface can rise to - or above - ground level. Although having only a secondary influence, changes in the Mississippi's stage can also alter the groundwater table's vertical location. Consequently, all such factors must be considered when assessing the phreatic surface's active/passive impacts on HDD conduct.

POTAMOLOGICAL/GEOTECHNICAL ANALYSES

The Mississippi River's role in site development plus the in situ soil parameters relevant to HDD execution are defined by appendices B and C, the latter titled *Geotechnical Analysis*. Pertinent aspects are the river's alluviation characteristics and HDD's applicability to the site specific materials.

Potamology. Today, the Mississippi River near the project site is a **marginally unstable Class IV course** whose alluviation - both before and after flood protection levee construction - has been driven by *active channel incremental displacement*. While the stream will again encroach on your HDD site, such event is sufficiently far in the future as to be of no concern. Rather, what is important is that the Mississippi's past meandering generated the earth material stratification relevant to HDD implementation. Specifically, the river's pre-levee abandonment of the bendway reach now defined by Lake Carlisle fostered accumulation of the soils described in the previous section:

- the *Inchannel Fill* silt and sand
  overlain by the
- the *Bank Surface Sedimentation* clay "drying" crust.

Followon construction of the flood protection levee - besides curbing the profile development process - actually imparted some of the foundation soils' strength/density characteristics: weight of the embankment served to consolidate the underlying earth materials. Consequently, soil stratification to be negotiated by site-specific HDD - while of alluvial derivation - has been both naturally as well as artificially altered since deposition.

Geotechnology. In terms of conducting HDD, all presently discovered in situ geotechnical conditions are amenable to such process. Therefore, the project site presents an opportunity for a meaningful/practical examination of the construction technology as applied to an essentially granular profile.

GEOTECHNICAL RECOMMENDATIONS

For a "standard" HDD project, geotechnically-based recommendations are usually made regarding HDD geometry, procedural conduct, etc. The research nature of this project, however, obviates the need for such suggestions: you are configuring the HDD installation to satisfy objectives other than placement of a "permanent" pipeline obstacle crossing. Per this understanding, geotechnical recommendations are limited to supporting the study's conduct rather than accomplishing the construction's execution.

Basically, only three recommendations present themselves:

- Groundwater table behavior monitoring - via use of the inplace piezometers - should begin well before HDD construction start-up.

- Design of the post-construction forensic excavation, including planning for dewatering, should be based on the soil strength/permeability parameters stated in appendix C.

- Closure of the excavation should be according to Corps' standards for levee construction compaction.

Purpose of the former recommendation is to aid in accounting for non-HDD influences on phreatic surface performance - especially Mississippi River stage fluctuation and rainfall variation. The latter two recommendations are intended to help foster project safety and site restoration. In all these regards, we are available to provide additional geotechnical input as you deem necessary.

DAVID P. SAULS
REGISTERED PROFESSIONAL
ENGINEER
12769
STATE OF MISSISSIPPI

Very truly yours,

LOUIS J. CAPOZZOLI & ASSOCIATES, INC.

David P. Sauls

David P. Sauls

Charles W. Hair, III

DPS/lb

Enclosures: Appendix A, Geotechnical Support
Appendix B, Potamological Analysis
Appendix C, Geotechnical Analysis
Appendix D, Field and Laboratory Analyses
Sheet 1, Site Location/Vicinity
Sheet 2, Site Conditions
Sheet 3, Site Plan and Subsurface Profiles

LOUIS J. CAPOZZOLI & ASSOCIATES, INC.

GEOTECHNICAL SUPPORT

Overall aspects of the site-specific geotechnical efforts provided by us for this project are discussed in the paragraphs below. Followon enclosures - mainly *appendices B, C,* and *D* plus sheets 1 through 3 - present particulars.

Purpose. Geotechnical site characterization necessary for a pipeline installation's design/construction - especially if construction is to be via the HDD trenchless technique - first entails definition of the obstacle's geometry (width, depth, bank heights/steepnesses, etc.) and in situ soil stratification. Based on these determinations; a river obstacle's dynamic qualities, i.e. its' potential for horizontal meandering and vertical scouring, plus the intended placement methodology's site specific applicability/conduct particulars can then be evaluated. In short; knowledge of onsite soil material types, amounts, and conditions - in concert with site-area geological/topographical study results - enables:

- assessment of past, as well as the potential for future, river activity;
- determination of HDD's site specific feasibility and environmental impacts;
- selection of a crossing geometry to allow pipe placement in as amenable soil conditions as possible, with minimized environmental disturbance, while providing safety from alluvial activity disruption during the installation's intended life;
- development of data for tailoring pipe placement procedures and equipment to better economize site-specific construction; and
- conduct of analyses, i.e. levee slope stabilities, etc. - requisite to designing and implementing the crossing's tie-ins to the pipeline proper.

Under normal circumstances, use of such information in the crossing's design, permitting, and installation phases facilities accomplishment of the project's overall objective: efficiently placing/maintaining the pipe across the waterway obstacle. In the case at hand, however, research aspects are paramount. Consequently, projecting the Mississippi River's future alluviation is not as important as:

- defining the site's developmental mechanisms, both natural and artificial;
- generating subsoil parameters relevant to HDD planning and execution; plus
- forecasting various phenomena accruing from HDD performance in the site's constituent earth materials.

Additionally, placement of instrumentation to monitor in situ effects of HDD conduct is a "non-standard" facet of this particular geotechnical effort.

Constituency. The geotechnical data base consists of:

- Geological conditions, site development aspects, and generalized soil conditions from:

The Mississippi Geological Survey's
Geologic Map of Mississippi
1:500,000 scale
1969 (Reprinted 1985)

The Mississippi River Commission's
File No. MRC/2588 SH. 18-D
Geological Investigation
Mississippi River Alluvial Valley
December 1944

The U.S. Army Engineer Waterways Experiment Station's:

Technical Report No. 3-480
Geological Investigation of the Yazoo Basin, Lower Miss. Valley
Revised March 1968

Quaternary Geology of the Lower Mississippi Valley
1:1,250,000 approx. scale
1974

The USDA Soil Conservation Service's and
Mississippi Agricultural Experimentation Station's
Soil Survey of Issaquena County, Mississippi
Issued November 1961

- Area topography and waterway course locations taken from USGS maps:

1:100,000 scale:	Bastrop, Louisiana/Mississippi - 1982 Yazoo City, Mississippi/Louisiana - 1984
1:24,000 scale:	Lake Providence, Louisiana/Mississippi - 1982 Millikin, Louisiana/Mississippi - 1970 Mayersville, Mississippi/Louisiana - 1970 Whiting Bayou, Mississippi/Louisiana - 1970

- Observations from engineer visits by us in mid-March (onground) and mid-April 1996 (overflight);
- Data from our mid-May through late-June 1996 onland geotechnical field exploration, instrumentation installation, and laboratory testing efforts; plus
- site-specific topography from late May 1996 measurements by your surveyors.

Pertinent, data base details are in the followon enclosures.

Conduct. Support procedure involved our geotechnically analyzing the data base information relative to your HDD installation of two parallel, small-to-medium diameter, steel pipelines beneath a deactivated section of Mississippi River flood protection levee. The geotechnical work's overall scope and constituency were determined via various communications between us: January through March telephone conversations; our 19 and 29 March *Geotechnical Support proposal letters;* plus your 25 April *Request for Quotations No. DACW39-96-Q-1125.* Conduct authorization was your *Requisition Request No. W81EWF-6114-9943.* Overall auspices for geotechnical support execution was your *Purchase Order No. DACW393-96-M-1337* dated 13 May 1996. To facilitate the project's design, permitting and construction schedule; preliminary findings of our analyses - i.e. crossing schematics, site plan/profile data, piezometer configuration information, etc. - were presented to you as they were developed. Details of our as-executed efforts are described in *Appendix D.*

Overall constituency of this geotechnical support generally equates to numerous recent geotechnical investigations by us for others of river crossings in the southcentral United States. Particularly applicable are evaluations of the:

- Mississippi River main channel at several sites between Taft, Louisiana and Rosedale, Mississippi (ref. LJC&A files 81-39, 83-95, 83-178, 84-6, 85-210, 88-72, 89-91, 90-66, 91-54, 91-77, 92-65, and 95-35);
- Red River at locations from Shreveport to Red River Landing, Louisiana (ref. LJC&A files 81-90, 88-25, 88-114, 90-39, 92-56, 92-83, 93-97, 94-12, and 96-23); and the
- Atchafalaya River at positionings above Melville to below Morgan City, Louisiana (ref. LJC&A files 84-185, 85-80, 85-211, 89-80, 92-84, 92-88, 94-82, and 95-38).

In essence, the investigative format employed for your research project closely corresponds to the analytical methodology developed/applied by us on almost 300 new and replacement pipeline/telecommunications cable crossing site characterizations accomplished throughout the continental United States during the past fifteen years.

POTAMOLOGICAL ANALYSIS

The general and site-specific alluvial processes responsible for project-site stratification are discussed in this appendix. Illustrations are presented by sheets 1 through 3.

General. The function of any river - whether an alluvial system of continental proportions or an intermittent flow creek - is to move water and entrained earth material from its source to its mouth. In serving this purpose, the river not only *actively* establishes its bed/bank conditions but also *passively* responds to them. Consequently, central thrust of the potamological (river study) site evaluation phase is to understand the stream's *regimen*: the overall, as well as the site specific, manner by which the river's flow continually reconfigures/repositions its channel.

Controlling the regimen is the river's continual seeking of equilibrium between its entrainment capacity and its (natural as well as manmade) bed/bank material types and conditions. Defined as a stream's ability to pick up and carry a particular size and volume of earth material particles, *entrainment capacity* is proportional to *flow velocity*, i.e. current speed: a faster current is capable of picking up and carrying a larger size and/or greater volume of earth material particles than is a slower one. In turn, flow velocity is largely established by the river's *hydraulic gradient:* the horizontal distance from the stream's source to its mouth divided into the intervening vertical head loss. For the long term, i.e. during geological time spans, the hydraulic gradient will vary principally through alteration of the *vertical* component: worldwide sea level fluctuations (mainly due to global climatological factors, i.e. the onset/abatement of glaciation, etc.); epeirogeny (the overall mechanism by which the earth's crust is deformed/folded); plus orogeny (the various processes of mountain building, i.e. continental plate tectonics, volcanism, etc.) will generate hydraulic gradient changes. During the short term however, i.e. during human/pipeline corporation life spans, the hydraulic gradient is relatively fixed: sea level and land form positioning will normally remain constant during the time frame of pipeline design/operation interest. Therefore, the river will reconfigure itself in keeping with the established hydraulic gradient's *horizontal* component. Such reconfiguration has 2 forms: *gross channel relocation* and *active channel incremental displacement.* The first mechanism usually entails development of new, or reoccupation of old/abandoned, courses principally through *channel bending*. In the bending process, the river develops "meanders" which - as they mature (enlarge) - move downstream until eventual cut off. The second horizontal adjustment normally involves less dramatic active channel positional and/or configuration changes within the established gross channel. Accomplishment is via either:

- prolonged ablative attack of one bank combined with inchannel deposition (point bar building) on the opposite side, i.e. a mini-version of the bending process, or
- expansion of the course cross-section through generation of bank slope failures (landslides) "simultaneously" in both sides.

Of course, long term occurrence of incremental active channel displacement can eventually lead to gross channel relocation. In context, then, these 2 mechanisms produce the propensity for a river to meander across its floodplain. Evidence of such past activity are oxbow lakes and sloughs (water filled "cutoff" courses), meander scrolls (alternating curvilinear ridges and swales defining former channel edge locations), etc.

Also affecting the entrainment capacity via current speed alterations are flow volume departures from the norm, i.e. floods and droughts. These generate "momentary" entrainment capacity variations whose effects - expansion and contraction, respectively, of the channel cross-section - are not only responsible for stratigraphic anomalies (gravel "lenses" within finer grained alluvium, etc.) but may also evoke further alluvial response (course relocation due to scour hole filling or point bar deflection, etc.) during more normal flow periods.

The naturally heterogenous composition of a river's entrenchment conditions - i.e. variations in makeup of its' bed/banks - significantly influences the stream's regimen.

Differences in earth material types (clay, sand, gravel, rock, etc.); placement origins (aeolian, colluvial, alluvial, glacial, marine, etc.); and in situ conditions (loose, dense, consolidated, lithified, etc.) offer highly varying responses to a specific entrainment capacity. Likewise, natural phenomena (faults, etc.) and manmade features (conventionally installed pipelines, bridge piers, channel training devices, etc.) also contribute to establishing a river's overall, as well as site specific, regimen.

Considering maintenance of the entrainment capacity-entrenchment material equilibrium as the overall controlling factor, four generalized river obstacle conditions, i.e. classes, are apparent:

Class	Entrainment Capacity Character	Equilibrium Characterization Horiz.	Equilibrium Characterization Vert.	Generalized River Regimen
I	Increasing	Stable	Unstable (Penetrating)	Initial, or subsequent, stream development via mostly vertical scouring of either: - non-alluvially deposited/lithified earth material or - river sediment placed during a previous regimen. Single channel reaches are comparatively straight. Channel bank/alluvial valley walls are steep and form at compact, normally "V" shaped cross-section. New, hydraulically efficient distributaries of established rivers are examples of this relatively rare class.
II	Constantly High	Marginally Unstable	Stable	Stream has completed developing vertically and is in the process of horizontally expanding (broadening) its floodplain/alluvial valley floor. Reaches are generally braided, i.e. consist of many interconnected subchannels rather than a single course. Individual subchannels have flattened "U" shaped cross-sections. Entrenchment conditions usually consist of entrainment resistant alluvium (generally gravel or cobbles) over non-alluvial/lithified material.
III	Decreasing	Unstable	Marginally Stable (Filling)	Stream, in the process of filling (sedimenting) its previously formed floodplain/alluvial valley, is nominally in a single channel configuration. Since the river is assuming a meandering, i.e. bending, characteristic, channel sections through the relatively compact course consist of lopsided "V"s interspersed with flattened "U"s. Horizontal activity includes both incremental active channel displacement as well as gross channel relocation. In these regards, the lopsided "V" cross section point, i.e. the thalweg, is on the outer - "attacking" - side of a bend while the flatter leg constitutes a "building" point bar. The intervening flattened "U" sections are termed channel crossings since the thalweg is transitioning between opposing sides at such locations. Vertical activity involves reduced flow channel "filling" coupled with flood induced scour penetration.

Class	Entrainment Capacity Character	Equilibrium Characterization Horiz.	Vert.	Generalized River Regimen
IV	Constantly Low	Marginally Unstable	Marginally Unstable	Stream is entrenched in a mature floodplain consisting of alluvial sedimentation offering varying degrees of entrainment resistance. The mostly single channel course consists of well defined meanders. Overall, the cross-section is non-compact: an *active channel* (usually water filled) is contained within a *gross channel* constituting a meander belt. Natural levees - channel paralleling ridges of comparatively coarse grained alluvium sedimented from overbank flood waters - generally define the gross channel's edges as well as some positionally stable active channel bounds. Course cross-sections equate to those described for Class III. Horizontal activity by the active channel involves both incremental lateral migration as well as course bending within the gross channel confines. Repositioning/relocation of the gross channel - via either incremental migration or bending - plus re-occupation of former alignments is also possible. Vertical activity, usually confined to within the gross channel's limits, can result from: - thalweg penetration at an attacking bank, i.e. scour on the outside of a meander bendway. - channel crossing migration, i.e. displacement (usually downstream "descent") of the channel crossing along a comparatively straight reach. Resulting in periodic bed "deepening"/filling against opposing banks, this phenomena is "triggered" by localized entrainment capacity aberrations: current speed changes due to "steps" from vertical faults intersecting the channel, placement of manmade channel training devices, tributary head cutting (channel deepening) processes evoked by an hydraulically "improved" tributee, etc. - eddy currents induced by "hard point" anomalies (manmade or natural) in the bed/banks. On balance, this class is probably the most prevalent of the 4 categories, especially in the continental United States.

Since a river is a linear feature, at any given time a particular stream can exhibit most - if not all - of these classes at some point(s) along its course.

Mississippi Regimen. The Mississippi River and its tributaries, in draining a large percentage of the North American continent east of the Rocky Mountains, constitute one of the premier alluvial systems on the face of this planet. Length of the main channel, from source to mouth, approximates 2300 miles. In the vicinity of this project site, i.e. in the river's lower valley, chief alluvial characteristic is the channel's meandering, convoluted nature: airline distance between your site and the river's mouth on the Gulf of Mexico is almost 300 miles while the channel's centerline length is about 66 percent greater. In keeping with this, the hydraulic gradient (flow slope) is relatively flat: head (water pressure) loss between the site and

the Gulf of Mexico normally ranges between 80 and 110 feet. These overall factors in conjunction with:

- the in situ geotechnical conditions (themselves products of both alluvial and marine deposition mechanisms)
- the works of man

show the Mississippi at this site to be a **marginally unstable Class IV river** whose past, unconstrained alluviation is responsible for the in situ stratification's natural constituency.

Marine clay of Eocene - early Tertiary Period - age constitutes the soil bounding the alluvial valley. Such material dates from the time when a shallow sea - the *Mississippi Embayment* - covered the region as far north as present-day Memphis, Tennessee. Worldwide periods of lowered sea level, caused by periodic glacier formation, imparted the clay's significant characteristics: high strength and density plus slickensided prefractures. Chiefly responsible for these features were the loss of buoyancy and followon desiccation which developed when such soil emerged above the depositing water's surface. Reduced sea levels also allowed the river to vertically scour into the emplaced clay. Primary result is that Marine clay defines the absolute limits of past horizontal and vertical alluvial activity.

As intervening periods of glacier melting occurred, with consequent rises in sea level, the Mississippi and its local tributaries started filling the previously scoured valley with alluvial sedimentation. Significantly, the latest cycle of such process - beginning several thousand years ago and ongoing today - has so thickly covered the confining Marine soils with alluvium that the underlying "foundation" material will not be a factor in this project. In line with this, physical evidence - in the form of the *Carlisle Lake* abandoned course, the bank surface's pronounced meander scrolling, plus the site-specific levee set-back being subdrilled - reveals the river's continuing propensity for scouring into/meandering across its alluvially sedimented valley floor floodplain. That occurrence rate of such activity is not rapid is indicated by comparatively large natural levees bordering the present day Mississippi. Since natural levee development is a function of flooding, horizontal as well as vertical size largely depends on the number of over-bank episodes taking place from/at a particular course location. Consequently; the broad, well developed natural levees on both sides of the existing channel reveal local area repositioning - and the resultant "natural construction" of alluvially sedimented soil strata - was/is a relatively slow process.

The greatest present day *short term* influence on alluvial activity at this site is manmade channel "stabilization" for navigation assurance/flood prevention. Requirement for such management stems from the United States Congress's 2/3 century old mandate to the US Army Corps of Engineers for maintaining the Mississippi in its existing alignment. Initially, mandate exercise in the river's lower reaches involved coordinating the "tie-in" of various manmade flood protection levees/levee segments which had been "independently developed" during the preceding $2^1/_4$ centuries. Followon mandate accomplishment was/is embodied in a series of "5 Year" construction plans detailing the Corps' intention to perpetuate the alluvial "status quo" in both this site's immediate vicinity as well as elsewhere along the river's course. During the past half century, human efforts have had two basic effects. First, a series of upstream locks and dams - built across the Mississippi and its major tributaries - has reduced localized gradients and thereby incrementally "stilled" the current. A significant percentage of naturally entrained soil particles has thus been "scrubbed" from the flow. In fact, recent studies indicate that - due mainly to such mechanism's effects on the tributary Missouri and Arkansas Rivers - the Mississippi's contemporary particulate burden is roughly half the former maximum. Second, localized construction - bank revetting, flood protection levee maintenance, navigation channel dredging, etc. - has minimally constrained the river to a series of hydraulically efficient, relatively straight/narrow, side-hardened course segments both above and below the reach to be traversed by this pipeline. In concert, all such actions have both (momentarily) stabilized the river's course plus generated an enhanced entrainment capacity. The latter outcome's principle effect is alteration of the previously existing equilibrium between the river and its "natural" entrenchment materials: in essence, and especially during high flow "flood" events, the channel

is now more rapidly transmitting "sediment hungry" water. Overall resultant is that the Mississippi is potentially more capable of changing - at an increased rate than before - its near-site channel section to accommodate a "new" entrainment capacity/bed-bank condition equilibrium. Leading mechanism for accomplishing this is the development of bankslope failures - i.e. *crevasses* - which oftentime necessitate levee deactivations and/or setbacks. Fortunately, planning by the Corps envisions continual maintenance - for the foreseeable future - of the now inplace channel constraint facilities. Consequently, the ongoing natural alluviation which originally developed the project site - and was later responsible for its' deactivated levee section - should not again directly affect the study area until well past the time-frame of HDD research interest.

<u>Site Development.</u> Considering the Mississippi's past/present configuration near the project site, in situ subsurface conditions stem largely from *incremental active channel displacement*. In particular, horizontal activity driven by such process has resulted in abandonment of the channel reach now forming *Carlisle Lake*. In turn, this has allowed formation of the Recent Alluvium *Topstratum* soils relevant to the levee's foundation support and, therefore, HDD conduct:

- *Bank Surface Sedimentation* (the crust of nominally desiccated, medium to stiff strength clay) transported by floods from the Mississippi's modern course plus

- *Channel Fill* (the underlying loose to firm silt and sand) plugging the Carlisle Lake abandoned reach.

The deeper Topstratum layering together with the *Ancient Alluvium Substratum* and Marine *Tertiary* materials are beyond the depth of this study's HDD installation.

Contributing to the foundation soils' characteristics is the man-made levee itself. Soil consolidation/densification brought about by long-standing imposition of the deactivated embankment's weight has markedly increased overall competency of both the Bank Surface Sedimentation as well as the Channel Fill's upper zone. Of interest is that the artificially placed - i.e. human transported and compacted - levee material is, in reality, natural Recent Alluvium borrowed from nearby pits.

In summary, formerly unconstrained Mississippi River alluviation is responsible for the earth material types in which the HDD test will be performed. Subsequent to deposition, natural and artificial influences - notably near-surface drying plus the compressive weight of the manmade levee - have combined to generate such soils' pertinent characteristics.

GEOTECHNICAL ANALYSIS

Quantitative/qualitative soil data provided by this appendix are intended to both support recommendations stated in the report's text as well as delineate the subsurface profile of sheet 3.

Parameters. For the major earth material types identified at this site, details relevant to HDD design and construction are:

	Man-Made Levee/Recent Alluvium								
	Clay*				Silt*		Sand*		
Parameters.	Soft	Medium	Stiff	Very Stiff	Loose	Firm	Loose	Firm	Dense
Unit Weight									
-Total (pcf)	100	110	118	118	115	119	113	117	127
-Buoyant (pcf)	38	48	56	56	53	57	51	55	65
Isotropic Permeability Coefficient k (cm/sec)	1×10^{-9}	1×10^{-9}	5×10^{-9}	5×10^{-9}	5×10^{-6}	1×10^{-6}	6×10^{-3}	1×10^{-3}	5×10^{-4}
Strength									
-Cohesion, $\bar{c}$ (psf)	250	700	1,450	2,250	–	–	–	–	–
-Friction Angle, $\bar{\phi}$ (degrees)	–	–	–	–	26	28	27	29	33
Stress-Strain									
-Youngs Modulus, E (ksf)	75	200	350	500	50	175	100	250	600
-Shear Modulus, G (ksf)	25	67	117	167	17	58	33	83	200
-Poissons Ratio μ	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5

* Classification applies to soils containing additional constituents; i.e. sandy clay, sandy silt, clayey sand, etc.

The laboratory data table and figures of *appendix D* provide specifics.

Directional Drilling Applicability. Considering past experience with trenchlessly constructed river crossings, most earth material types/conditions are amenable to a properly engineered HDD procedure. Pilot hole accomplishment - barring contact with natural or manmade obstructions (logs, gravel pockets, channel stabilization/bank support piling, sunken barges, inplace pipelines, etc.) - is possible in virtually all earthen materials. However, certain soil types/conditions foster pilot stem steering imprecision as well as impede hole enlargement reaming.

Dense, high percentage gravel layers/pockets will adversely affect steering precision and thereby forestall efficient job execution. Other sources of steering imprecision are:

- pilot stem "skipping" when transitioning from a loose/soft stratum to a dense/hard layer
- pilot stem uncontrollability during passage through extremely soft/loose soils.

Gravel will likewise hazard successful reaming. Since the individual gravel particles are too heavy for entrainment by the drilling fluid (mud), and since their tendency to rotate in place

prevents them from being broken up by the reamer bit, they must be physically displaced during hole enlargement. Normally, displacement is radially outward into voids formed by entrainment of finer grained (sand and smaller size) particles. Because naturally dense, high gravel percentage soils contain little entrainable material, insufficient voids are developed during pilot hole accomplishment to permit follow-on passage by the larger diameter reams (usually to 12 inches in diameter greater than the carrier pipe diameter). In instances where the pilot hole is sloped (i.e. penetrates the gravel stratum at an angle), gravel particle displacement longitudinally - due to gravity after dislodgement by the reamer - will occur. This is advantageous only if voids exist, or can be formed, in the soils at the hole's vertical curves, i.e. the crossing's sagbends. If such is not the case, displaced gravel will collect in these pilot hole "sumps" to form impenetrable blocks. One positive note is that alluvially derived gravel and sand feature *rounded particles* - i.e. relatively smooth, non-angular shaped grains - which are not likely to score/gouge the carrier pipe's protective coating.

Relatively strong, impervious clays are - from a mechanical standpoint - almost ideal HDD media. Unfortunately, boring through them - especially if slickensided prefractures are present - potentially generates drilling mud surface seeps.

Based on the foregoing criteria, detailed assessment of HDD feasibility at the project site yields:

Earth Material Type	Gravel Constituency Range, Percent by Weight	Drilling Applicability
Soft to very stiff strength clay.	N/A	Excellent. At depth penetration of a stronger layer - if not conducted at a sufficiently steep incidence angle and/or into a preformed slot - may result in the pilot stem bit "skipping" along the weak/strong interface. Strong clay is an almost ideal directional drilling material although possible slickensided prefractures may allow extra-bore drilling fluid migration.
Loose to dense silt or sand.	<5	Good to Excellent. Gravel constituency is inconsequential. However, some steering imprecision may result during passage through the less dense material. Drilling mud - with viscosity, pressure, and volume matched to conditions - is necessary for hole maintenance during reaming, especially in the less dense strata. Proper drilling mud lubricity in concert with the alluvially transported granular material's generally rounded particle shapes should minimize/prevent damage to the carrier pipe's protective coating.

In essence, this site offers earth materials which are highly suited to HDD. Even so, successful trenchless construction conduct will require astute engineering - i.e. judicious selection of the bore's geometry/routing plus innovative planning of the drilling's conduct - in conjunction with pragmatic expertise on the part of the installation contractor. For study purposes, the underlying Recent Alluvium's *Channel Fill* granular soils - i.e. the in situ silt and sand - are acceptable HDD media. Consequently, all points of sagbend curvature should be entirely within them. Such measure will foster HDD efficiency by facilitating pilot bore steering and pre-pullback reaming plus help preclude drilling mud surface seeps (see below).

Below-ground conduct of HDD at this site should be relatively easy and not overly expensive. Therefore, chances are excellent for successfully and economically installing via HDD an environmentally compatible, short length, small-to-medium diameter, pipeline placement that should effectively demonstrate various phenomena associated with trenchless construction technology.

Site Integrity. Crucial aspects of environmentally efficient HDD conduct are the site specific measures necessary to:

- prevent/minimize inadvertent drilling mud returns
- preserve groundwater quality
- maintain/restore ground surface integrity.

In essence, drilling the pipe beneath the levee embankment must not cause undue drilling mud flows onto the ground surface nor adversely affect quality of the groundwater contained anywhere within the subsurface profile. Likewise, the necessary equipment access and drilling conduct activities must not extensively nor permanently disrupt the right-of-way surface.

A particularly important aspect of drilling through clays - especially if high strengths and healed prefractures are present - is minimizing the incidence of inadvertent drilling mud returns. Fortunately, the project site's installation layer - pervious granular silt and sand - will likely intercept/prevent extra-bore drilling fluid migrations. Careful handling of the HDD process will also serve to preclude such phenomenon. Even so, inadvertent drilling fluid returns may not be entirely preventable near the pilot bore's ground surface penetrations: the rig-side entrance point and/or pipe-side exit location. The contractor should therefore be prepared to perform cleanup.

Regarding groundwater quality protection: since the area's phreatic surface is likely directly tied into the adjacent *Carlisle Lake* meander swale, surface water quality should govern groundwater quality throughout the zone of pipeline installation interest. Consequentially, HDD should not greatly impinge this factor. In the normal case, measures to help restore surface integrity - i.e. grout plugging the ends of the soil-pipe annulus, rigorously backfilling drilling handling/recovery pits, etc. - are desirable and may be administratively required by permitters. However, the research character of this particular project - especially in view of the post-construction excavation of the emplaced pipeline - obviates the need for most such actions.

Accomplishing the geotechnical aspects of site cleanup/restoration is important to mitigation of construction impacts on the site's natural and artificial features. Included in such measures are: judicious selection of the HDD surface penetration locations; scheduling construction during "dry" and/or low river stage months; plus restricting/minimizing in situ excavation activities. For this particular project, site integrity maintenance will largely entail judiciously/safely performing the thru-levee forensic excavation of the HDD pipe coupled with careful backfilling of the resulting pit.

Exercise/implementation of the foregoing measures - especially by reducing the requirements for clean-up and/or optimizing the manner by which it is accomplished - should be based on maximizing the data to be gained from this HDD placement while minimizing the potential for long-term site disturbance.

FIELD AND LABORATORY ANALYSES

As-executed particulars of the geotechnical site reconnaissance, field exploration/instrumentation, and laboratory testing programs performed by us for this project are discussed below. Bases for such efforts were our 19 and 29 March 1996 *Geotechnical Support proposal letters* together with your 25 April *Request for Quotations No. DAC W39-96-Q-1125.* Detailed findings are in the attached *table, figures, instrumentation schematic,* and *logs of borings.* Graphical displays of results are on sheets 1 through 3.

Site Reconnaissance. Visits to the project location - onground on 11 March and an overflight on 14 May 1996 - were accomplished by our engineer. Overall objectives were to observe river channel/project site conditions in order to better assess findings of our geotechnical field exploration efforts. Selected photographs taken during the visits are presented on sheets 1, 2, and 3.

Reconnaissance expenses entailed travel in LJC&A's single engine, retractable landing gear airplane. Requisite ground transportation was provided by you.

Field Exploration/Instrumentation. For this project, six soil sample borings - numbered 1 through 6 - were accomplished by our drill crew/equipment during the period 14 through 16 May. Following sampling, each boring was converted into a piezometer for measuring groundwater pressure. Individual drill sites were pin-pointed by your survey personnel based on positioning established during our various communications and the onground visit. Furthermore, your personnel were onsite to coordinate access and observe/direct our operations. Approximate as-constructed locations of the boreholes/piezometers are graphically depicted on sheet 3.

Due to proximity of an active flood protection levee; you coordinated field work conduct with the *Mississippi Board of Levee Commissions* as well as the *Vicksburg District, US Army Corps of Engineers.* Such administrative actions were handled prior to our drill crew's arrival onsite.

Full depth advancement of each 4 inch nominal diameter boring was via the rotary washbore method. Footage was measured from the surrounding work surface: either the levee crest or the adjacent ground level. Termination of an individual boring was after penetrating to between the 30 and 60 foot depths. Such vertical extent helped ensure exploration of conditions impacting HDD. Stratification encountered by the borings did not require extensive measures for hole maintenance - i.e. inordinate amounts of drilling mud, full depth casing, etc. were not necessary.

Borehole sampling, in accordance with applicable ASTM specifications, was of two types. High quality undisturbed specimens - suitable for laboratory strength testing - were obtained by hydraulically pushing a 30 inch long, 3 inch O.D., thinwall steel Shelby tube into the ground a distance of 24 inches per sample. Classification samples were extracted via the Standard Penetration Test (SPT). This entailed driving a 24 inch long, 2 inch O.D., steel spoon into the ground with blows from a 140 pound hammer falling 30 inches. The resulting penetration resistance was the number of blows required to advance the sampler 12 inches after first seating it for 6 inches. Regardless of the method employed, though, sampling of all borings was the same: on 5 foot centers and/or at change of strata to within 10 feet of termination and then continuously.

Upon sampling completion, each boring was over-reamed for conversion into a piezometer. Each such device consisted of a pneumatic pore pressure transducer set at a predetermined depth and connected - via tubing - to the surface. Filter sand-packing the transducer/plugging-sealing the borehole was then accomplished to complete the installation

and prevent vertical flow-channel development. In the latter regard, grout make-up was the same Portland cement-bentonite mixture normally used to re-seal "standard" geotechnical exploration boreholes.

Tabularized field work particulars are:

			Depth	Piezometer Data						
Boring Number	Total Depth (Feet)	Continuous Sampling (Feet)	Factor >50 Ft. (Feet)	Serial Number	Tip Depth (Feet)	Tubing Length (Feet)	Borehole Closure (Feet)	Readings @ Test Pressures 5 PSI	50 PSI	100 PSI
1	30	10	--	46607	25	50	30	4.95	50.10	100.19
2	30	10	--	46606	28	50	30	4.92	50.20	100.29
3	60	10	10	46608	58	75	60	4.99	50.23	100.31
4	60	10	10	46609	55	75	60	4.65	50.20	100.24
5	30	10	--	46604	30	50	30	4.89	50.17	100.31
6	30	10	--	46605	28	50	30	4.80	50.20	100.12
Totals	**240**	**60**	**20**				**240**			

Detailed boring logs plus a schematic of the piezometer installation are attached.

Exploration expenses by LJC&A entailed: mobilization/demobilization travel of our equipment; 6 crew hours installing piezometers; plus 3 days crew living/local-area travel to and from the site. Support expenses involved purchase of the piezometers and ancillary equipment from *Slope Indicator, Inc.* of Bothall, Washington.

Laboratory Testing. Immediately upon recovery, each sample was field classified by our technician and then prepared for transport to our Baton Rouge, Louisiana laboratory. There, all undisturbed cores were lab classified and then subjected to strength plus unit weight/moisture content testing. SPT classification samples were analyzed for grain size distribution. Afterwards, specimens selected by you underwent Atterberg limit determination evaluation. In sum, laboratory efforts encompassed: 13 unconfined (U), and 13 unconsolidated drained triaxial (Qd), compression tests (each with a unit weight/moisture content check); 47 dry sieve analyses; and 11 Atterberg limit determinations. As per the borehole sampling techniques, all laboratory test procedures conformed to appropriate ASTM standards.

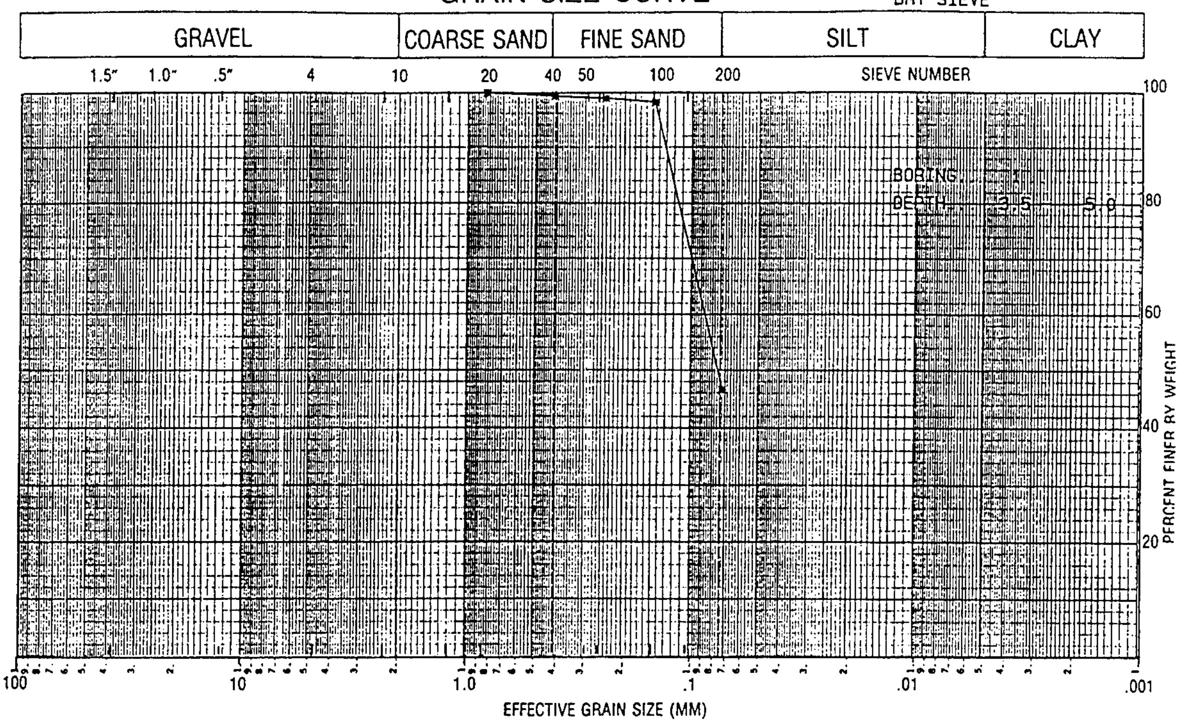
Compression testing, both unconfined as well as triaxial, yielded subsoil shear strength information. Unit weight/moisture content checks, the sieve analyses, plus the Atterberg limit determinations provided more precise subsoil classifications than obtainable through field methods. Taken together, findings of all such testing were used to assess/quantify the subsurface stratigraphy's origins and relationships to HDD. Finalized laboratory test results are presented by the attachments.

Attachments: Table 1, Laboratory Data
Figures 1 through 47, Grain Size Curves
Piezometer Schematic
Logs of Borings 1 through 6

EMBANKMENT SUBDRILLING STUDY — LABORATORY DATA — TABLE 1 — FILE NO.: 96-56

BORING NO.	DEPTH FEET	MOIST %	UNIT WEIGHT WET PCF	UNIT WEIGHT DRY PCF	ATTERBERG LIMITS LL	ATTERBERG LIMITS PL	ATTERBERG LIMITS PI	COMPRESSION TEST TSF	COMPRESSION TEST % STRAIN	PRESSURE-KSF START	PRESSURE-KSF FAIL	TYPE FAILURE	TESTING
1	0 - 2	25	119.9	96.0				1.50	6			MULTIPLE SHEAR	U
1	13 - 15	26	123.5	98.1				1.26	3	0.82		BULGE	QD
1	18 - 20	24	122.8	99.0				1.90	7	1.09		BULGE	QD
1	23 - 24				15	15	0						AL
1	25 - 26				19	19	0						AL
2	0 - 2	16	115.0	98.9				0.70	4			MULTIPLE SHEAR	U
2	8 - 10	33	111.5	83.7				1.31	4	0.52		BULGE	QD
2	13 - 15	33	119.6	90.3				0.94	10	0.82		YIELD	QD
2	23 - 24				14	14	0						AL
2	28 - 30	38	105.5	76.2				0.73	9			MULTIPLE SHEAR	U
3	0 - 2	6	117.7	111.5				2.71	5			MULTIPLE SHEAR	U
3	3 - 5	15	107.5	93.8				0.40	6	0.23		BULGE	QD
3	8 - 10	30	118.4	91.4				1.64	7			MULTIPLE SHEAR	U
3	13 - 15	43	92.5	64.8				0.07	10			YIELD	U
3	18 - 20	21	109.5	90.6				1.00	10	1.09		YIELD	QD
3	44 - 45				19	19	0						AL
3	51 - 52				22	19	3						AL
4	0 - 2	9	121.4	111.7				2.00	6			MULTIPLE SHEAR	U
4	3 - 5	16	125.2	107.5				0.44	2	0.23		BULGE	QD
4	8 - 10	26	107.2	85.2				0.28	10	0.52		YIELD	QD
4	13 - 15	19	123.1	103.2				0.24	5	0.82		BULGE	QD
4	18 - 20	23	119.3	96.9				2.65	10	1.09		YIELD	QD
4	23 - 25	41	111.5	78.9				1.35	4			MULTIPLE SHEAR	U
4	28 - 30	33	109.6	82.2				1.17	8			MULTIPLE SHEAR	U
4	44 - 45				22	22	0						AL
4	49 - 50				21	18	3						AL
5	0 - 2	14	108.9	95.6				0.76	8			MULTIPLE SHEAR	U
5	8 - 10	30	108.7	83.8				1.24	10	0.52		YIELD	QD
5	13 - 15	27	113.3	89.4				0.87	10	0.82		YIELD	QD
5	23 - 24				15	14	1						AL
5	27 - 28				20	19	1						AL
6	0 - 2	12	112.7	100.3				0.85	5			MULTIPLE SHEAR	U
6	3 - 5	22	124.9	102.0				1.73	5			MULTIPLE SHEAR	U
6	8 - 10	33	108.5	81.8				0.19	3	0.52		BULGE	QD
6	21 - 22				18	18	0						AL
6	23 - 24				18	18	0						AL
6	28 - 30	36	114.2	83.9				0.53	10			YIELD	U

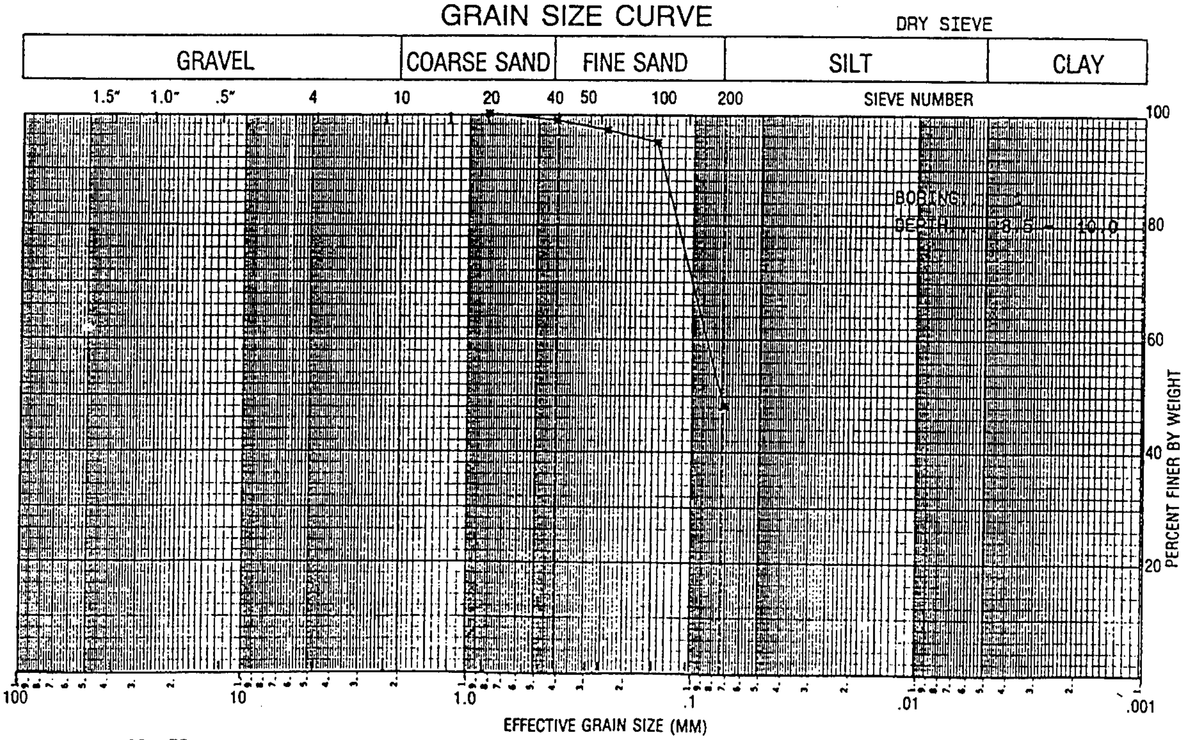
GRAIN SIZE CURVE



FILE 96- 56
FIGURE . . 1

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers

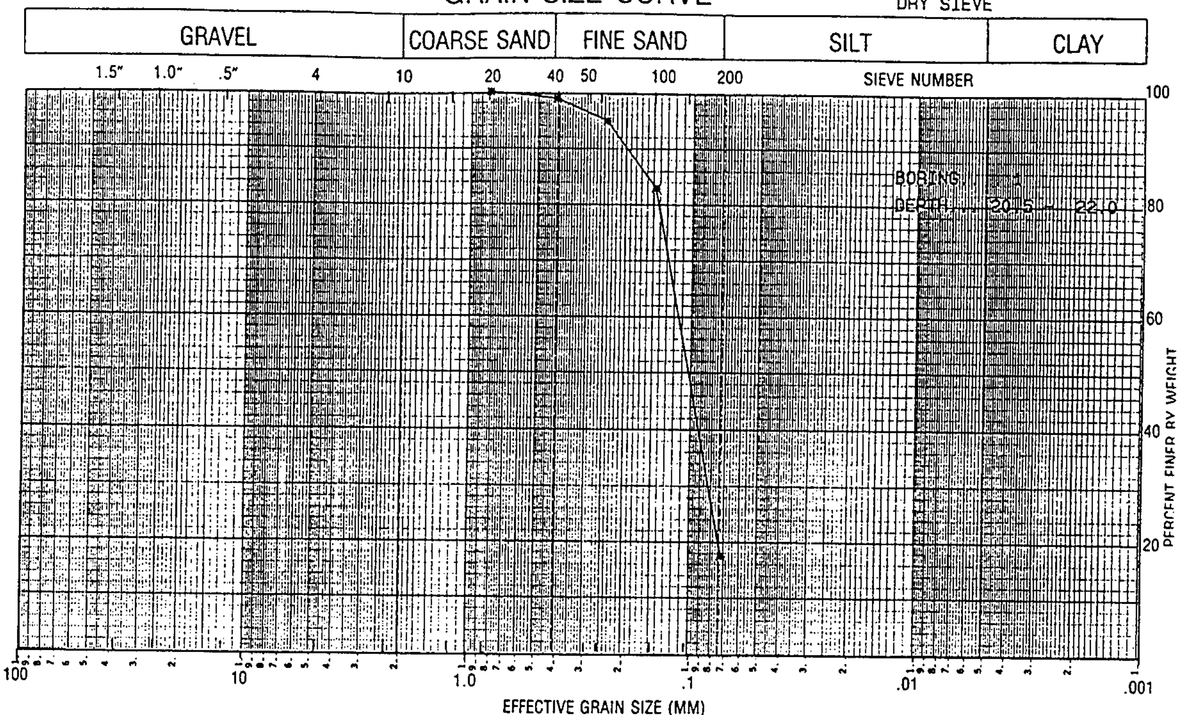
GRAIN SIZE CURVE



FILE 96- 56
FIGURE . . 2

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers

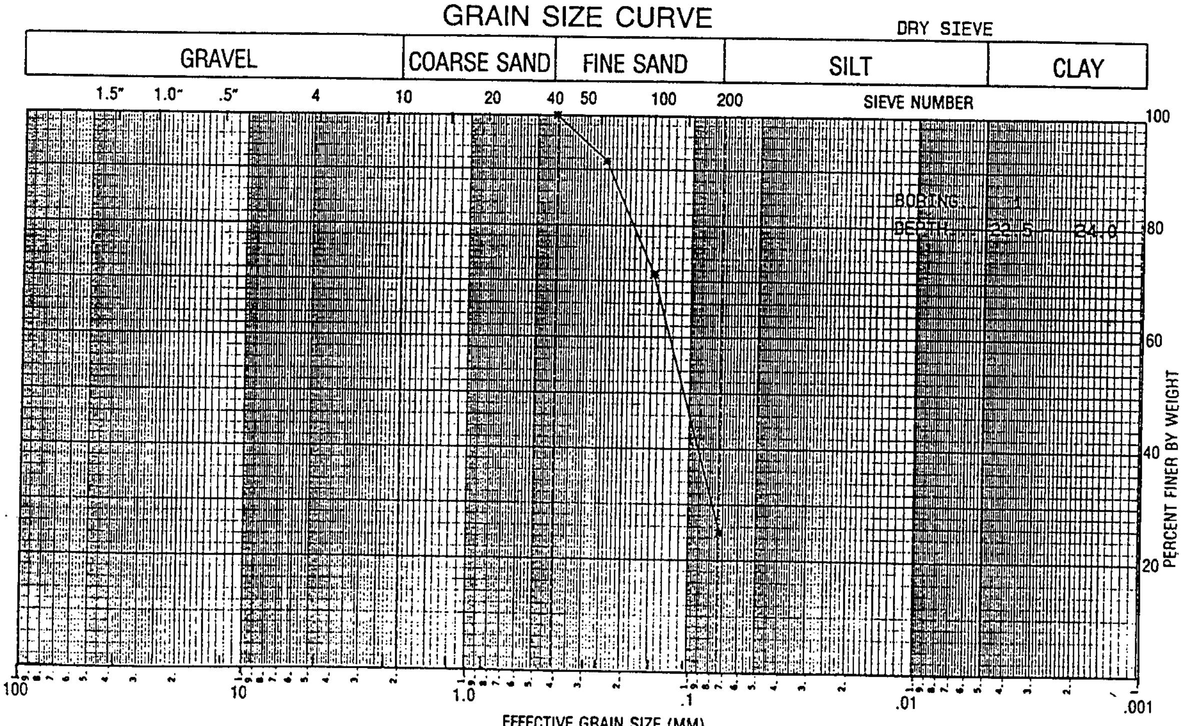
GRAIN SIZE CURVE



FILE 96- 56
FIGURE .. 3

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers

GRAIN SIZE CURVE

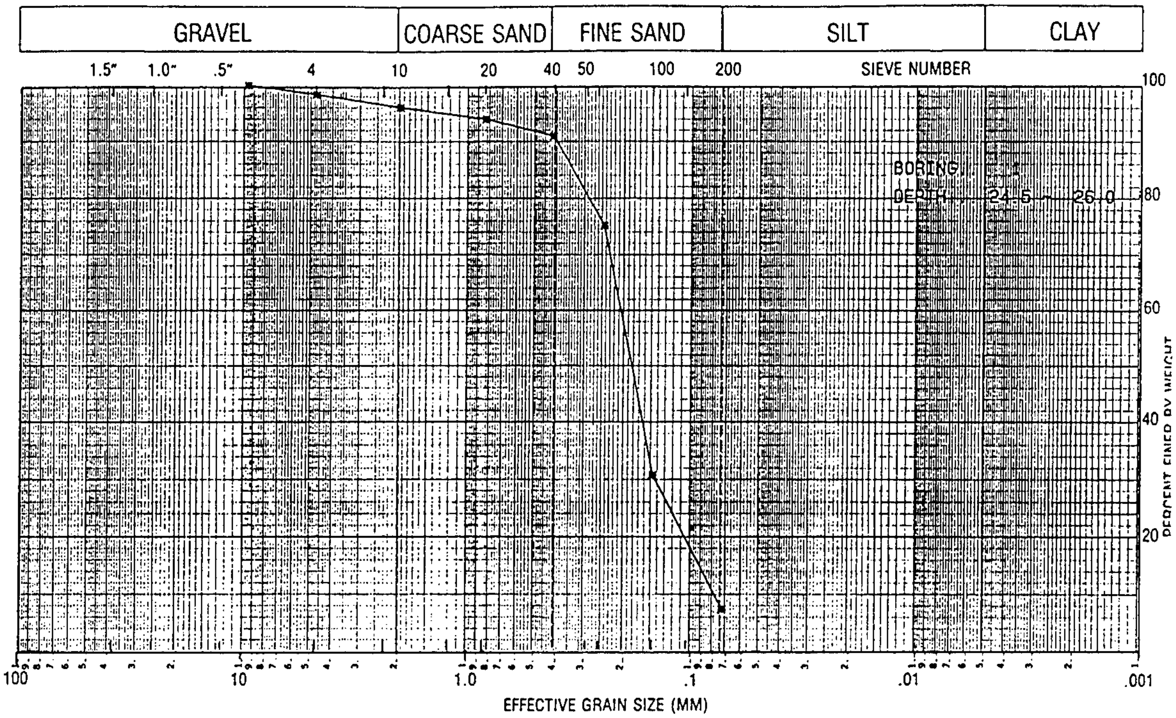


FILE 96- 56
FIGURE .. 4

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers

GRAIN SIZE CURVE

DRY SIEVE



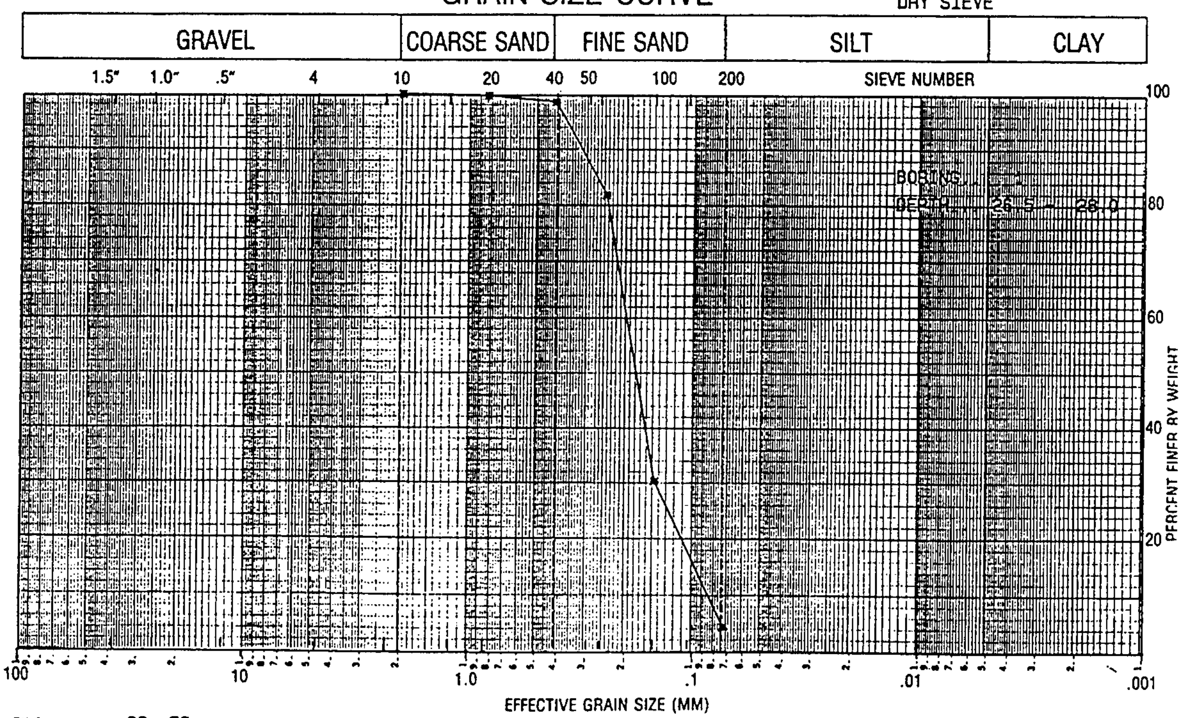
FILE 96- 56

FIGURE . . 5

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers

GRAIN SIZE CURVE

DRY SIEVE

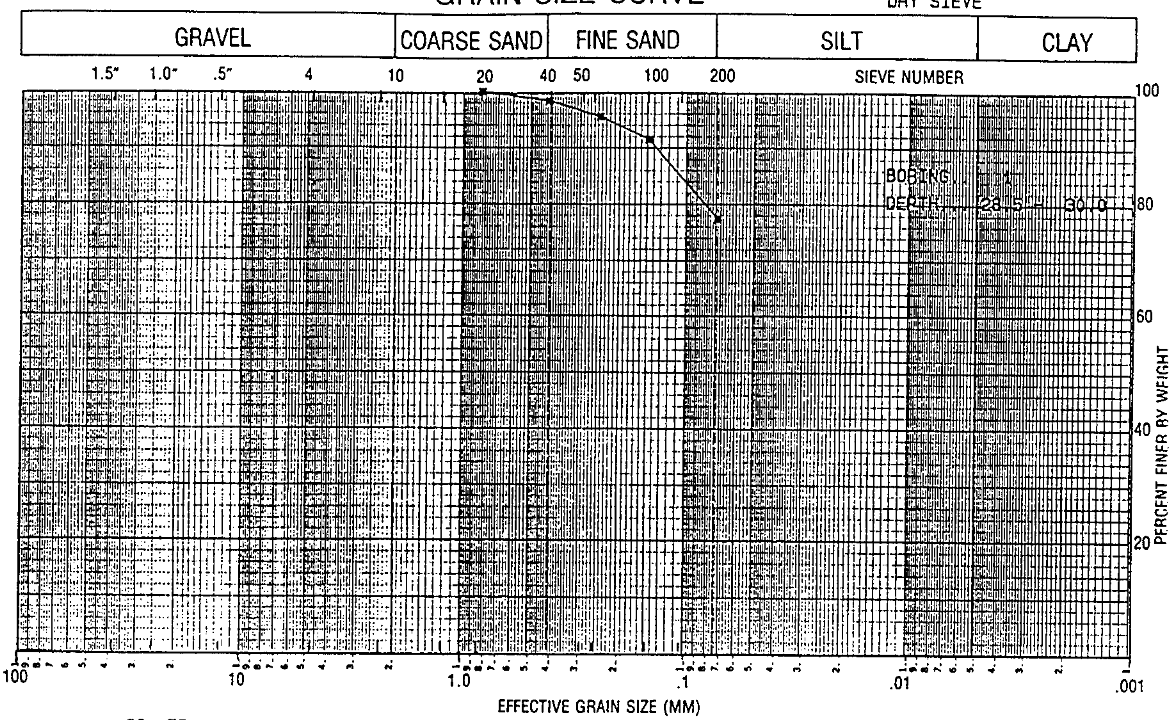


FILE 96- 56

FIGURE . . 6

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers

GRAIN SIZE CURVE

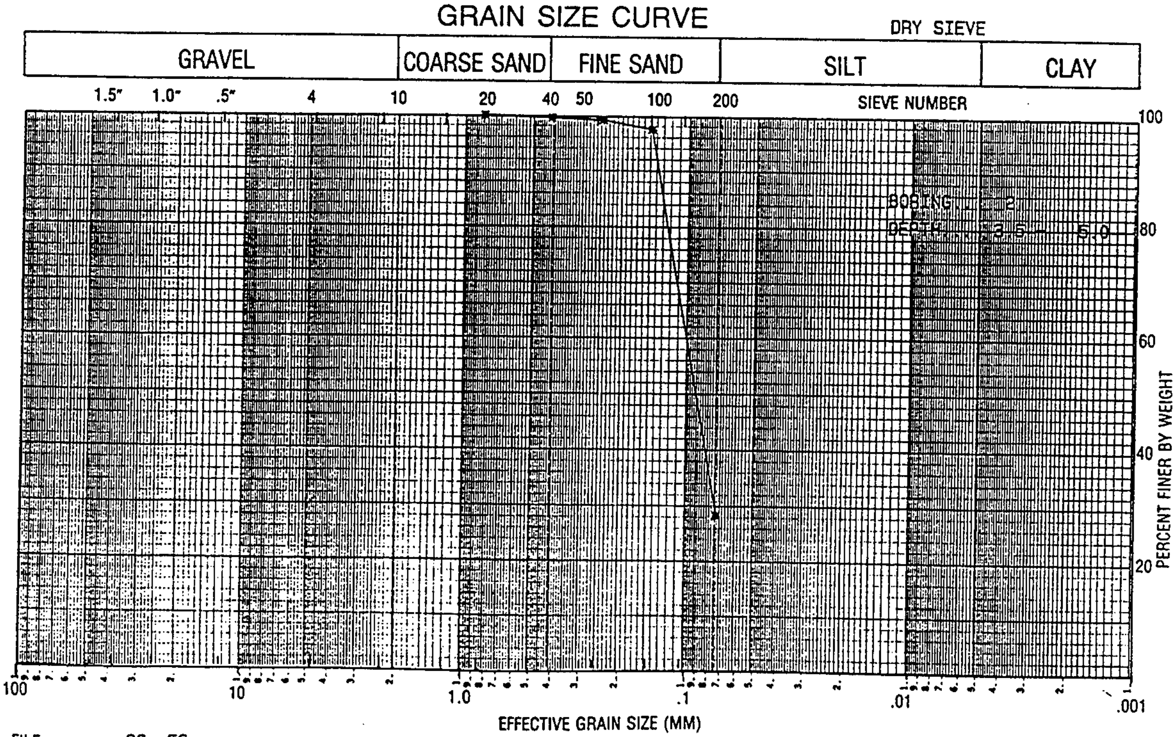


FILE 96- 56

FIGURE . . 7

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers

GRAIN SIZE CURVE



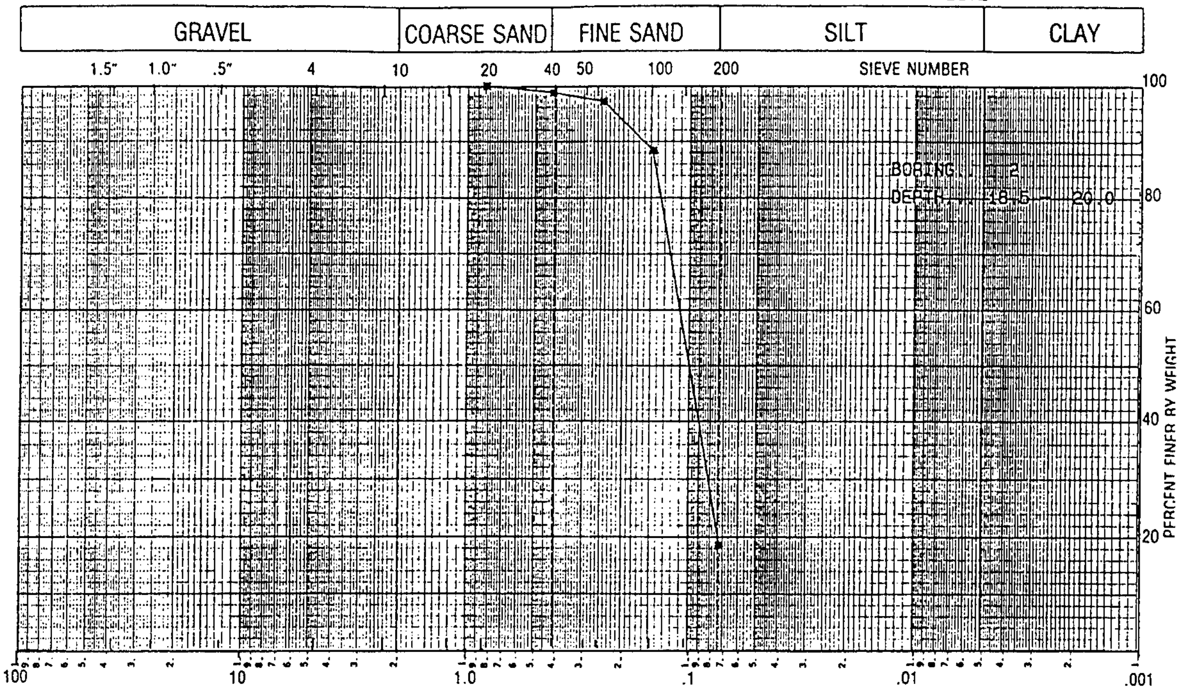
FILE 96- 56

FIGURE . . 8

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers

GRAIN SIZE CURVE

DRY SIEVE



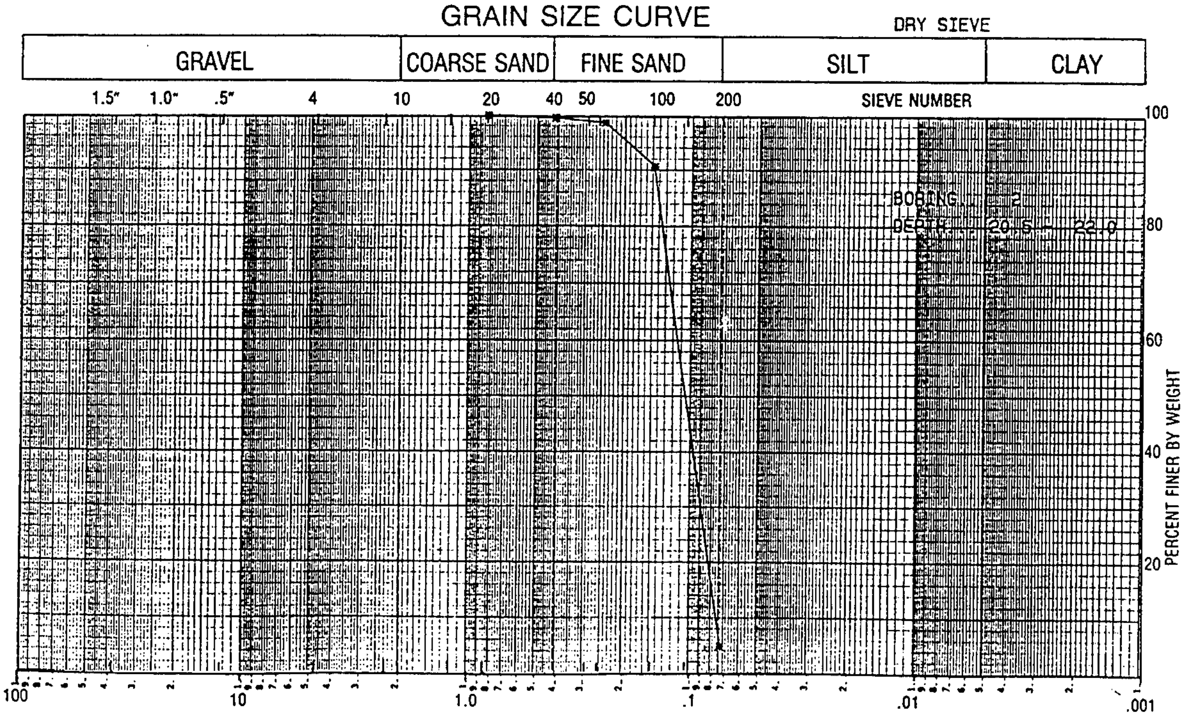
FILE 96- 56

FIGURE . . 9

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers

GRAIN SIZE CURVE

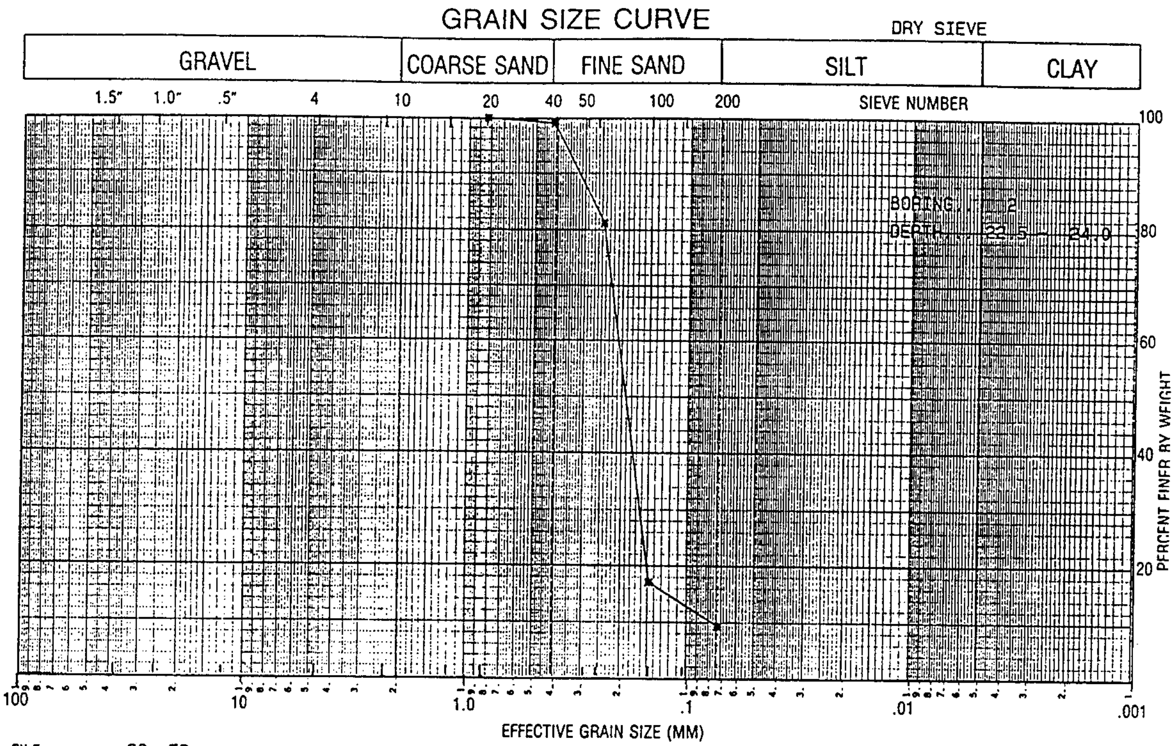
DRY SIEVE



FILE 96- 56

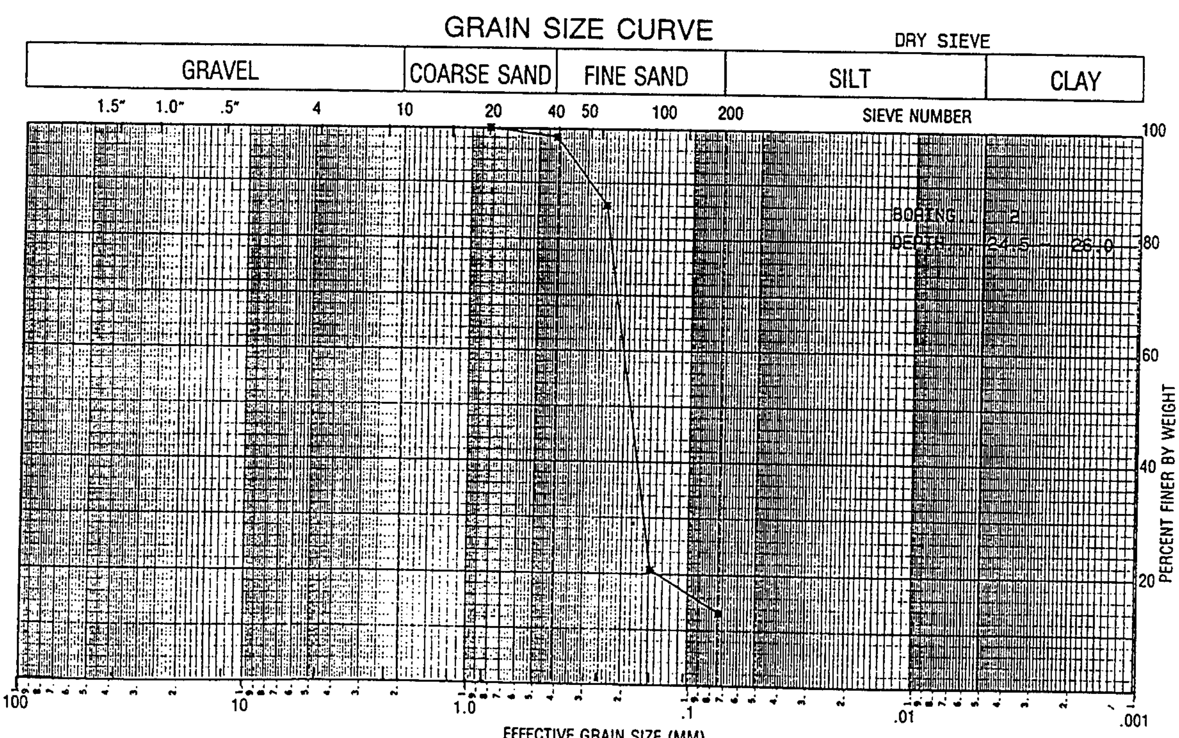
FIGURE . . 10

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers



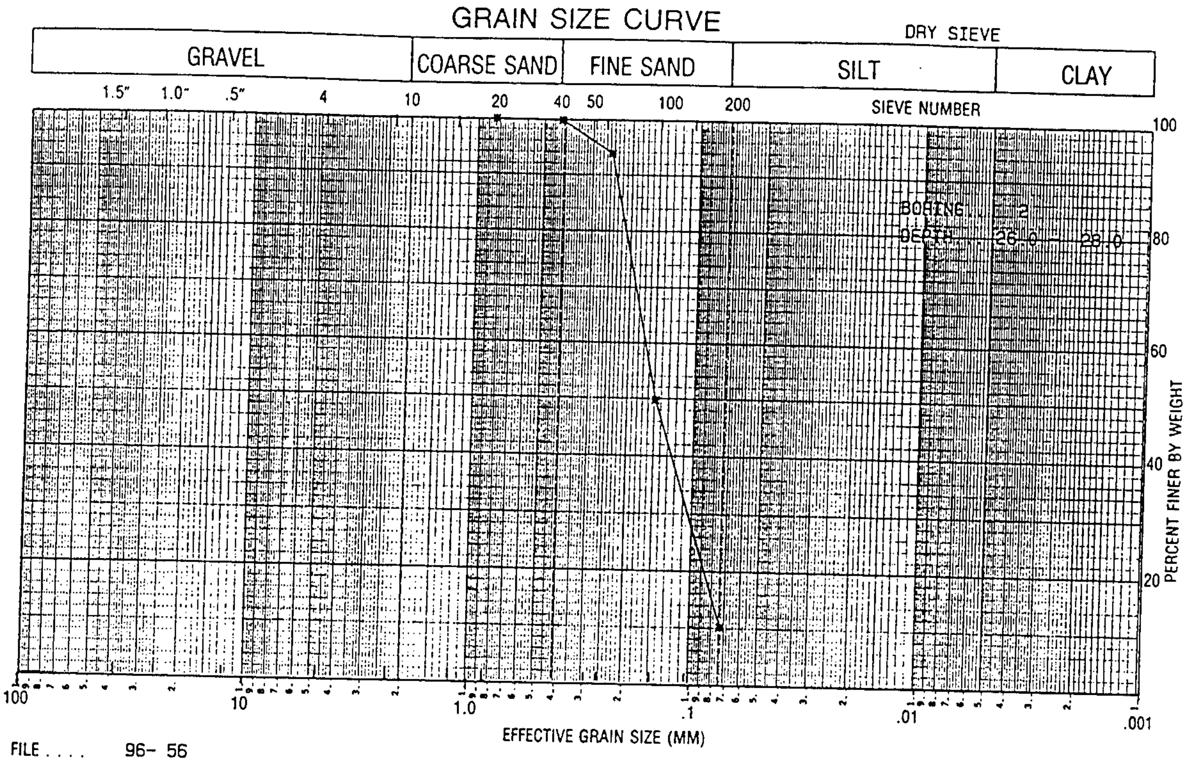
FILE 96- 56
FIGURE . . 11

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers



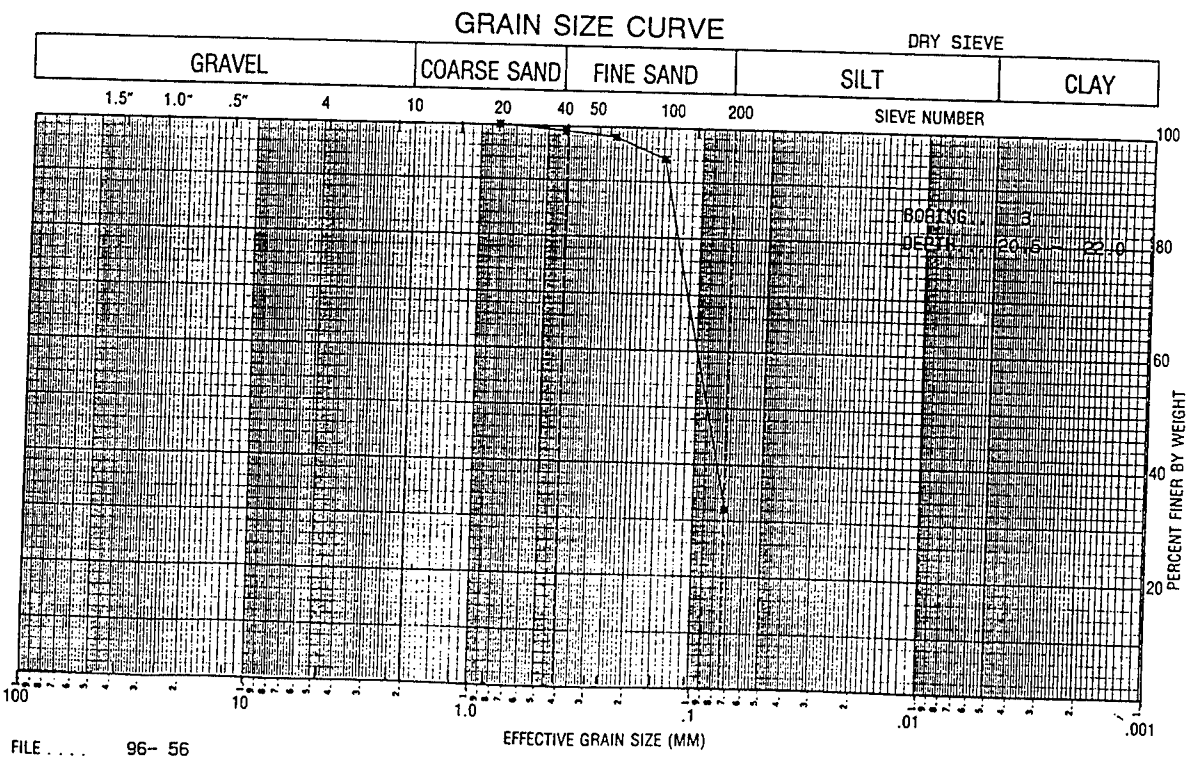
FILE 96- 56
FIGURE . . 12

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers



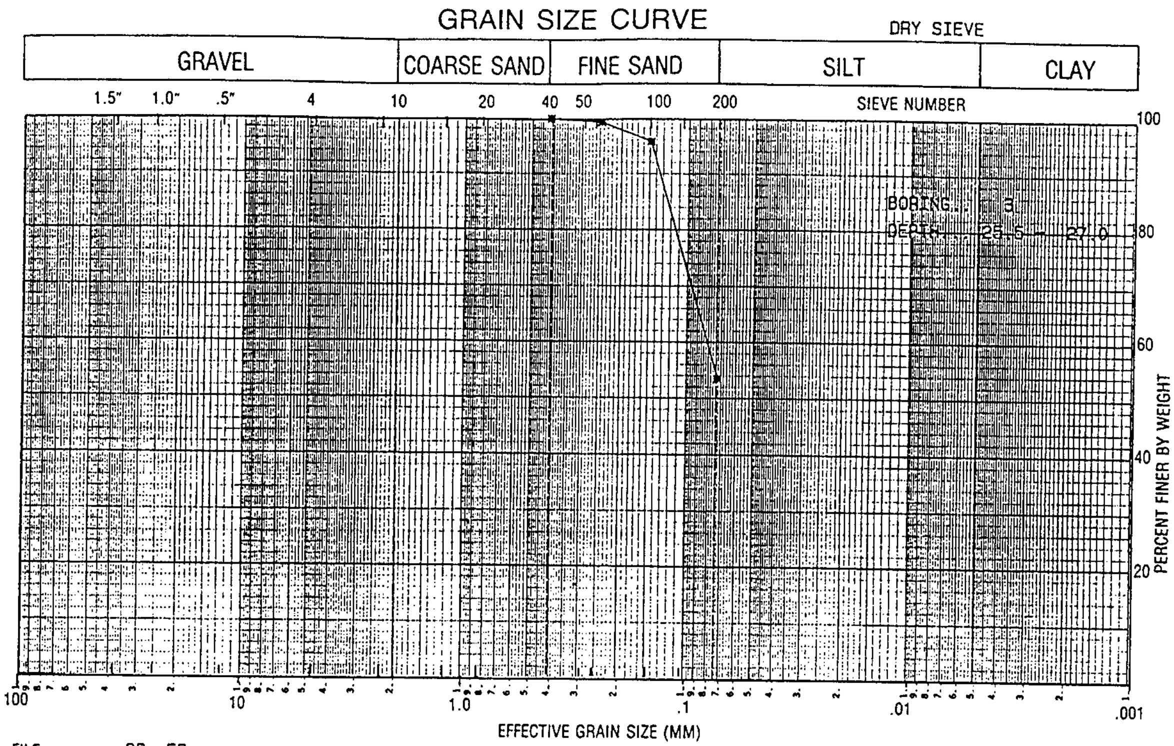
FILE 96- 56
FIGURE .. 13

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers



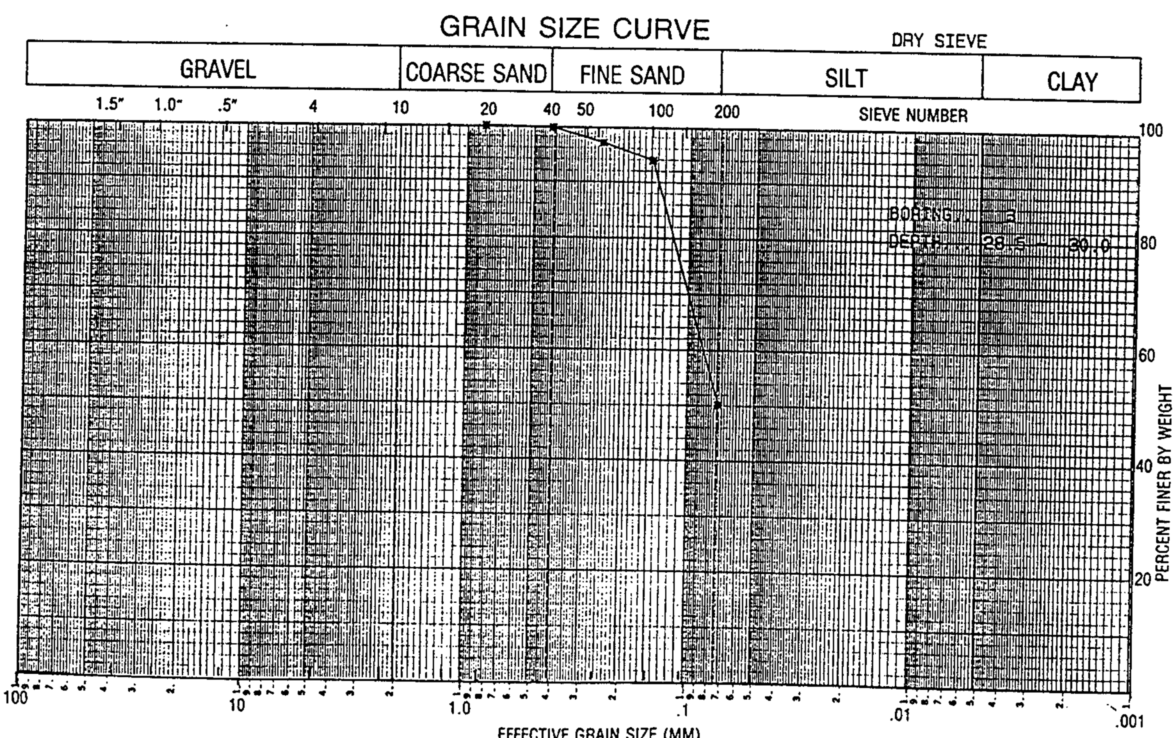
FILE 96- 56
FIGURE .. 14

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers



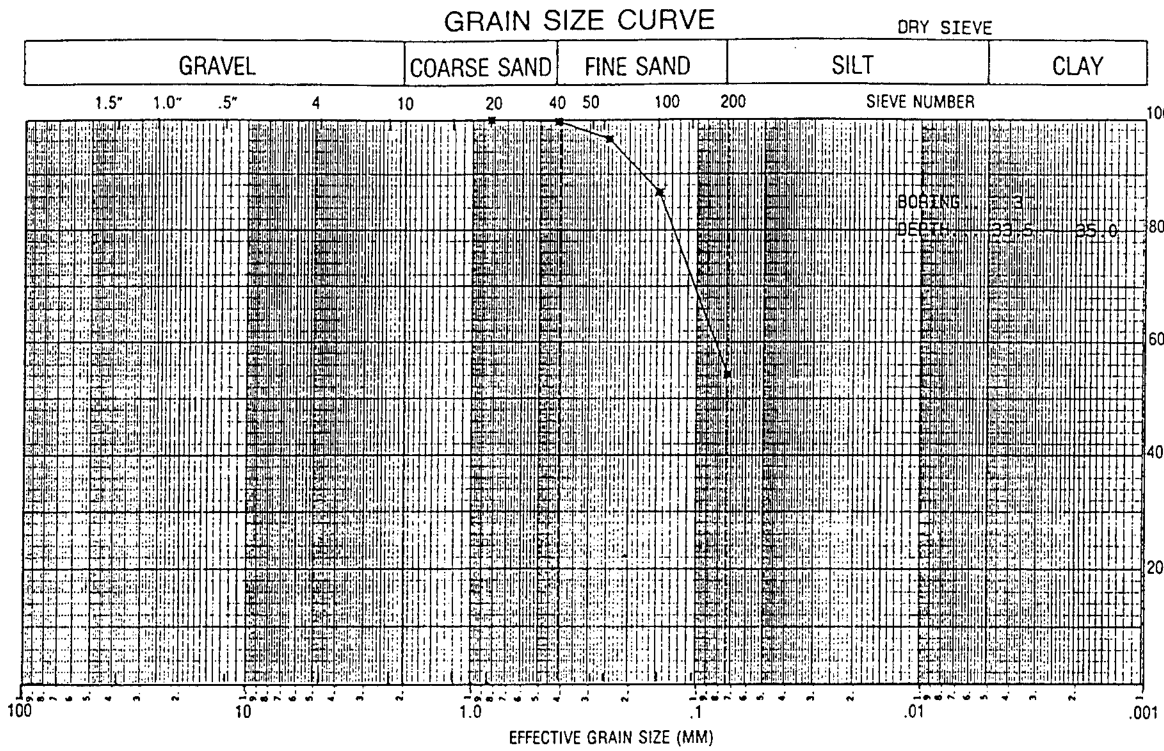
FILE 96- 56
FIGURE . . 15

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers



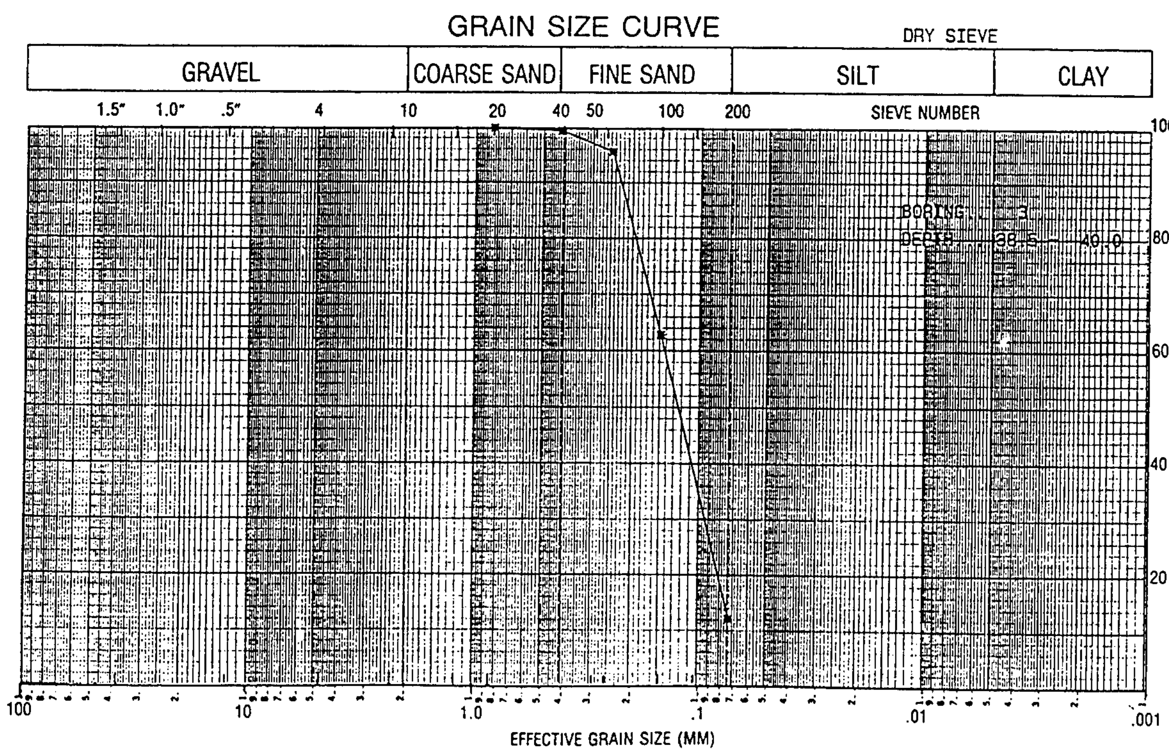
FILE 96- 56
FIGURE . . 16

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers



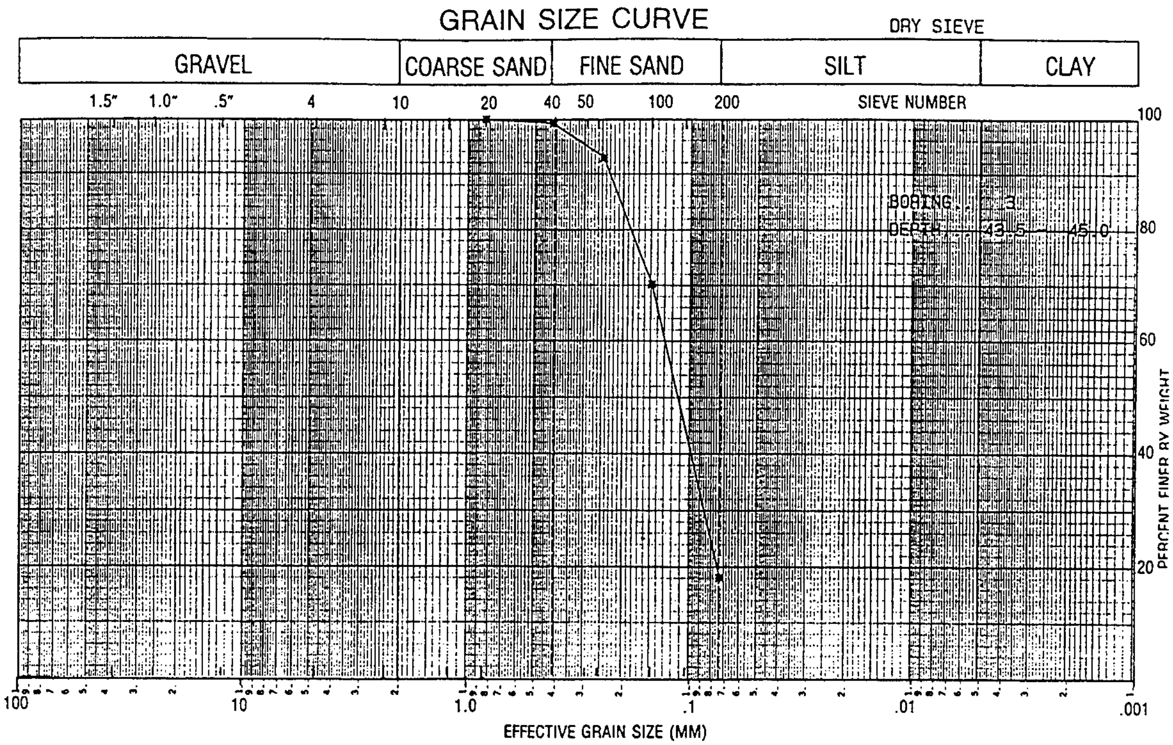
FILE 96- 56
FIGURE . . 17

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers



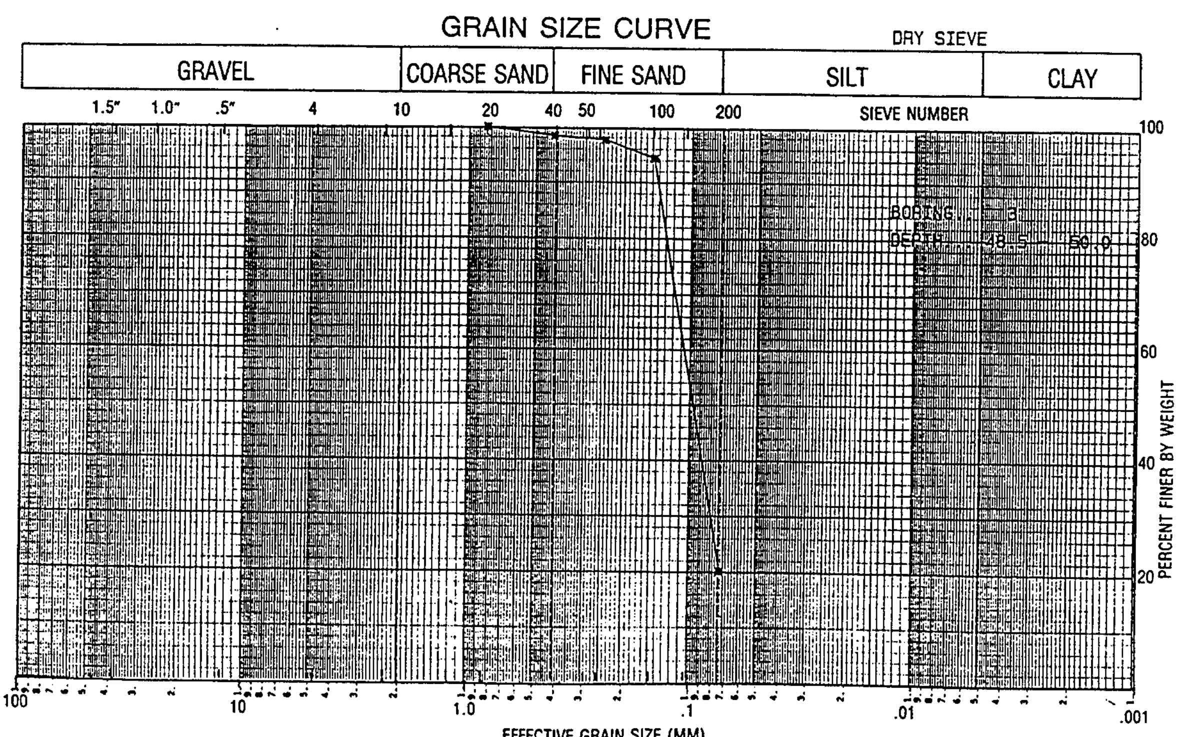
FILE 96- 56
FIGURE . . 18

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers



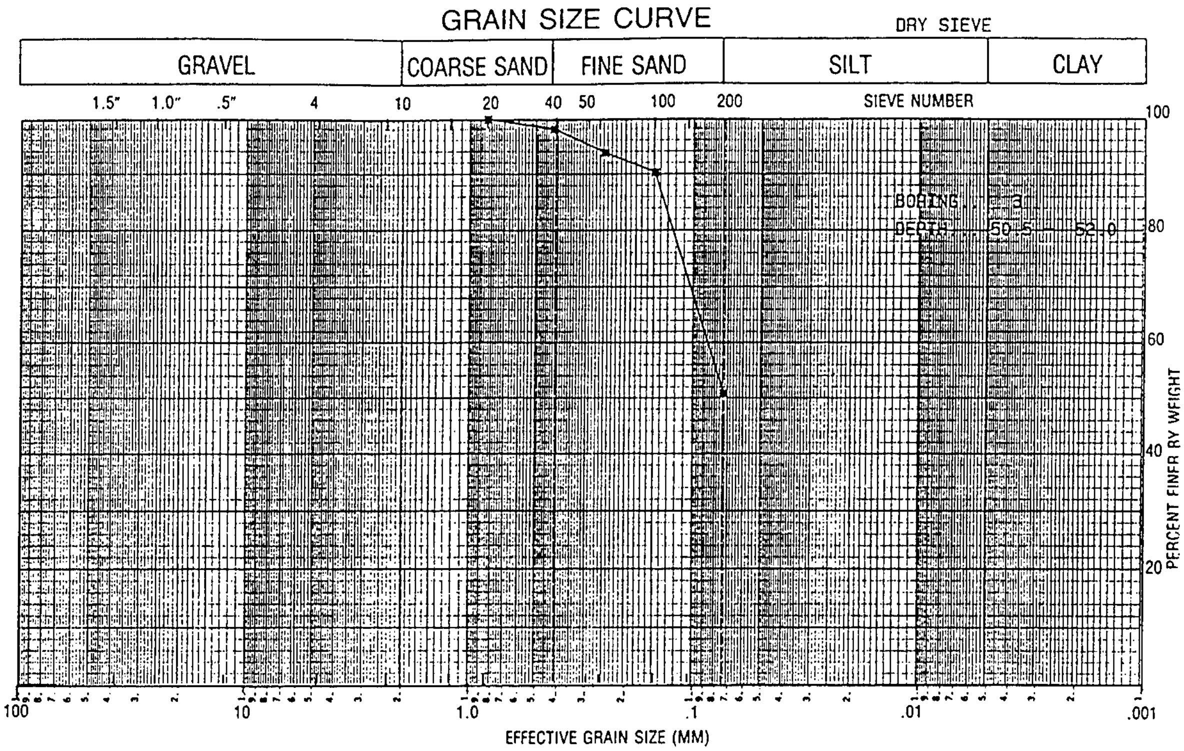
FILE 96- 56
FIGURE . . 19

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers



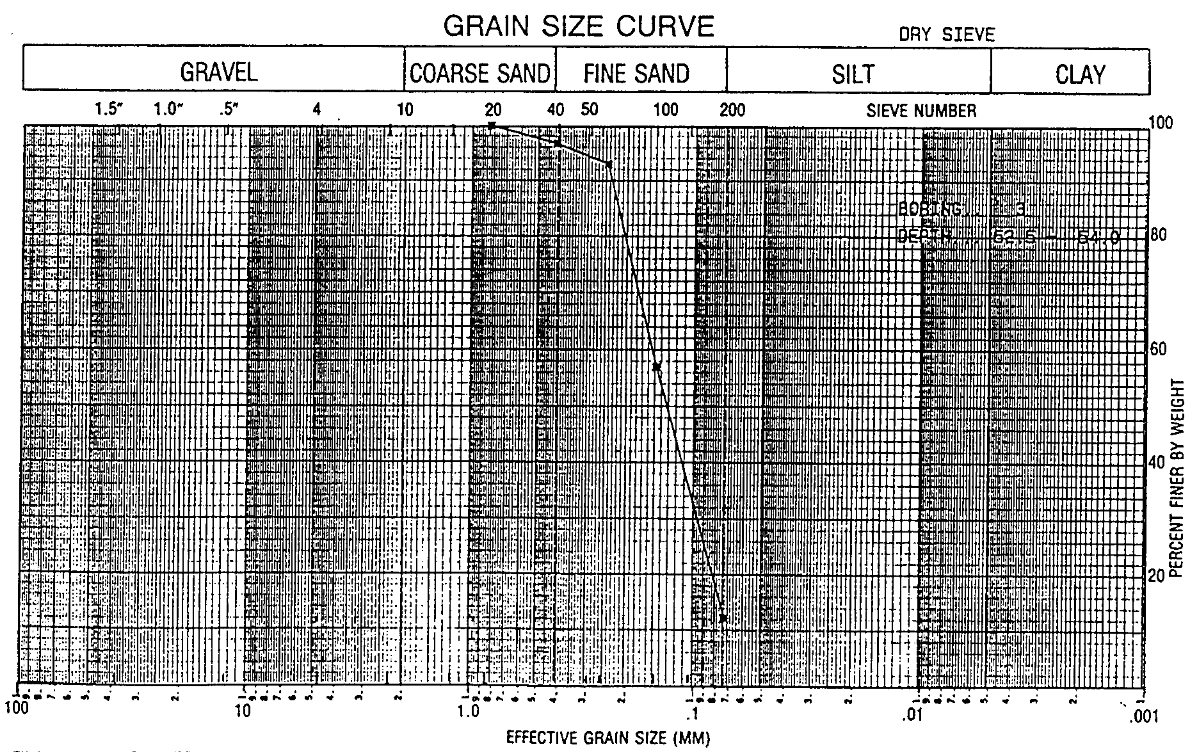
FILE 96- 56
FIGURE . . 20

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers



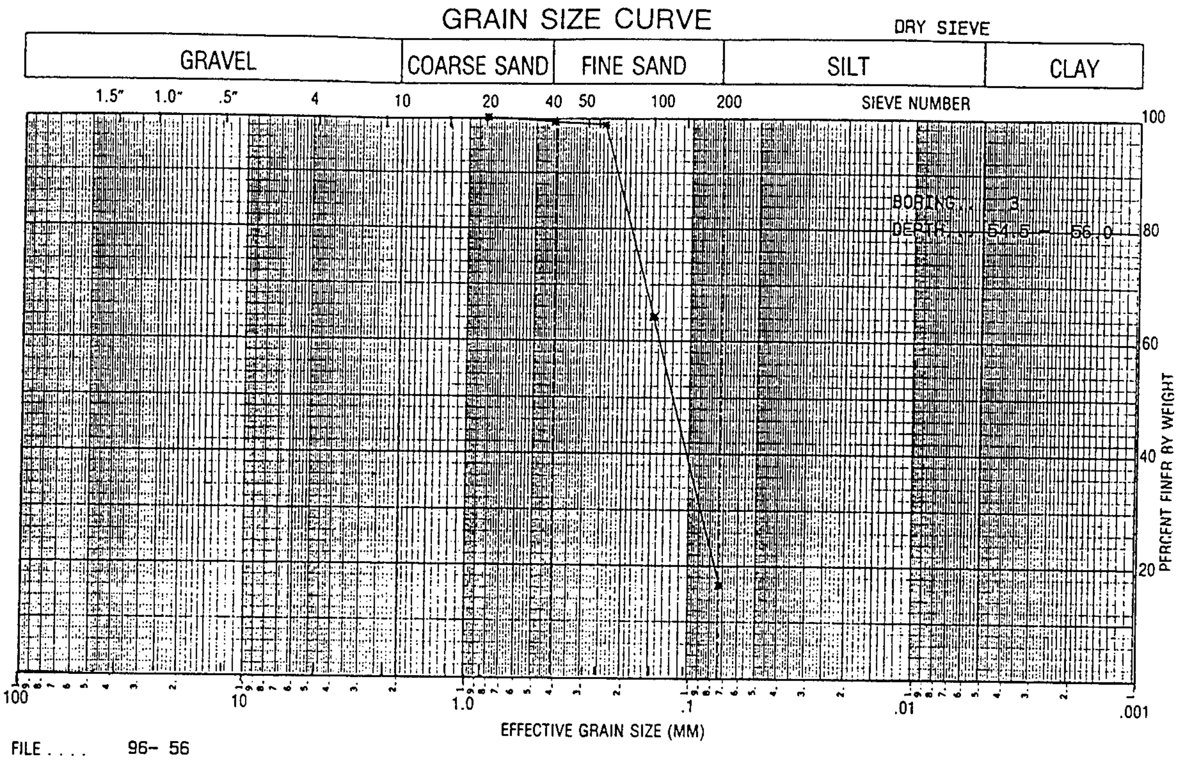
FILE 96- 56
FIGURE . . 21

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers



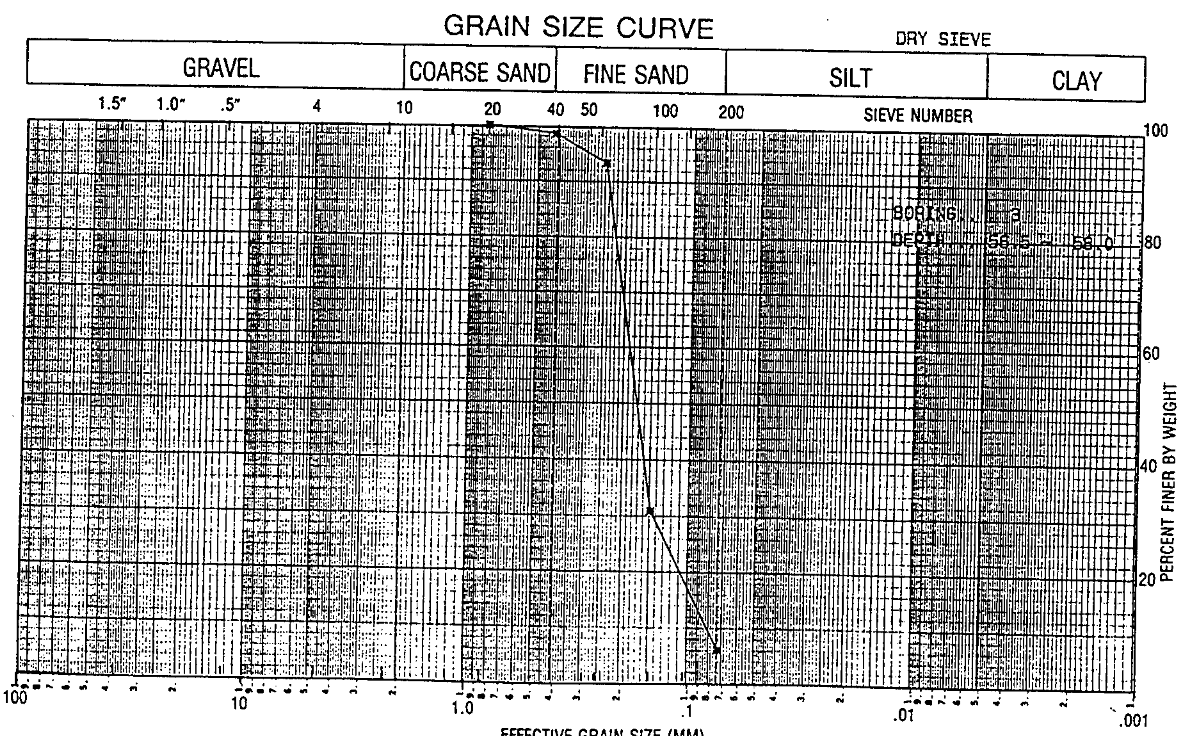
FILE 96- 56
FIGURE . . 22

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers



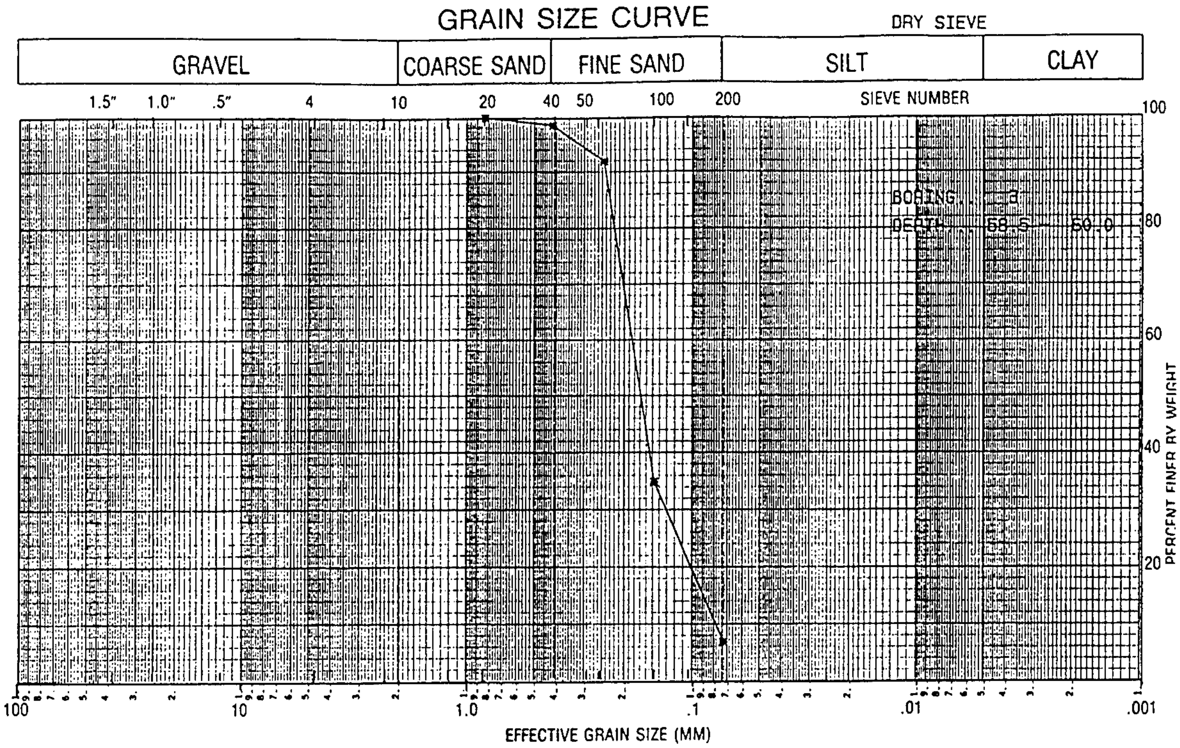
FILE 96- 56
FIGURE . . 23

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers



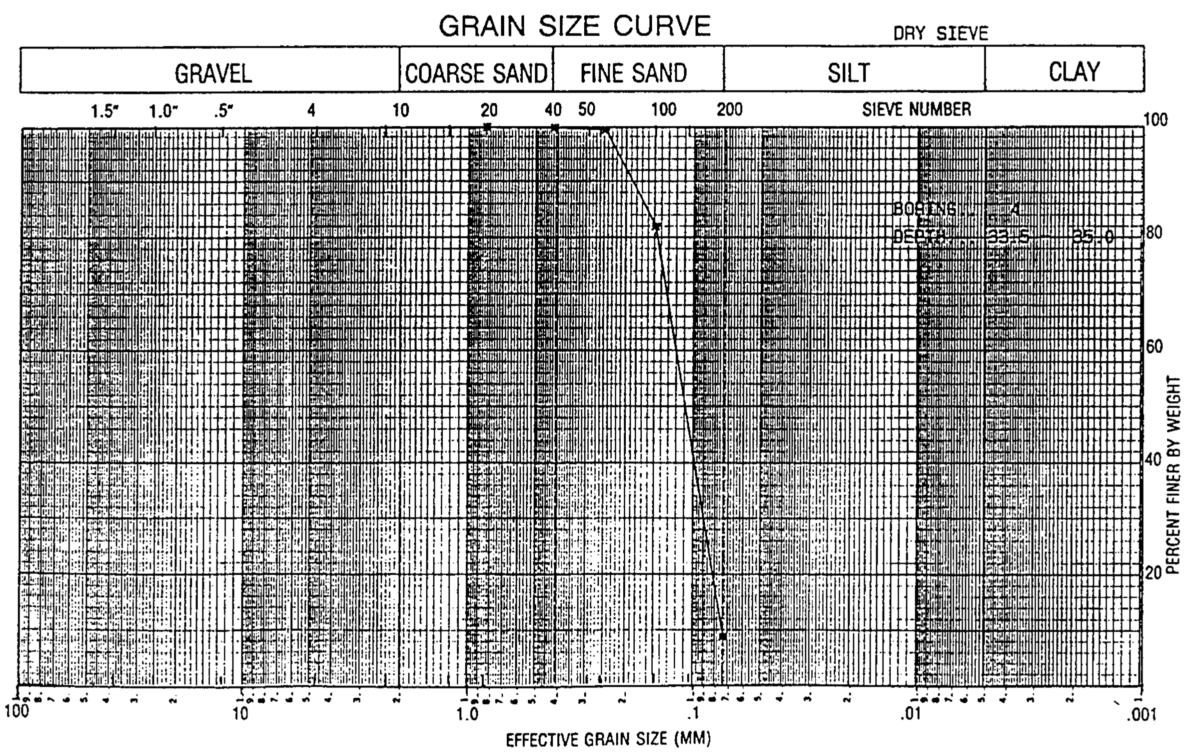
FILE 96- 56
FIGURE . . 24

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers



FILE 96- 56
FIGURE .. 25

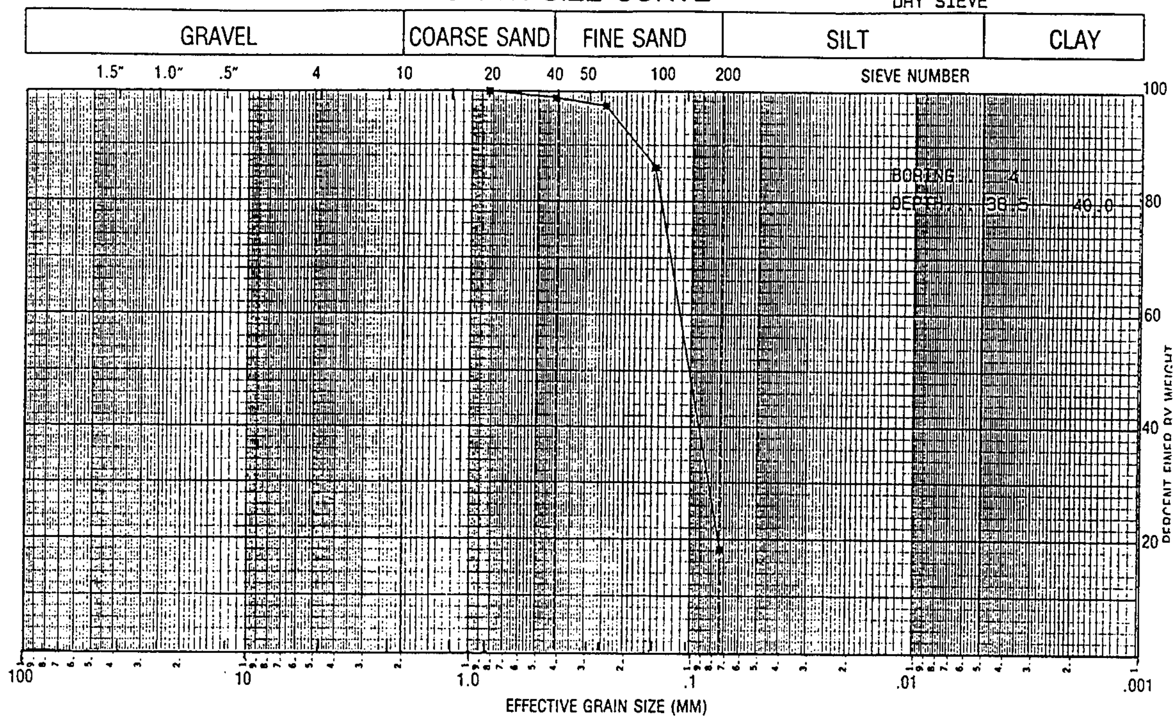
Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers



FILE 96- 56
FIGURE .. 26

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers

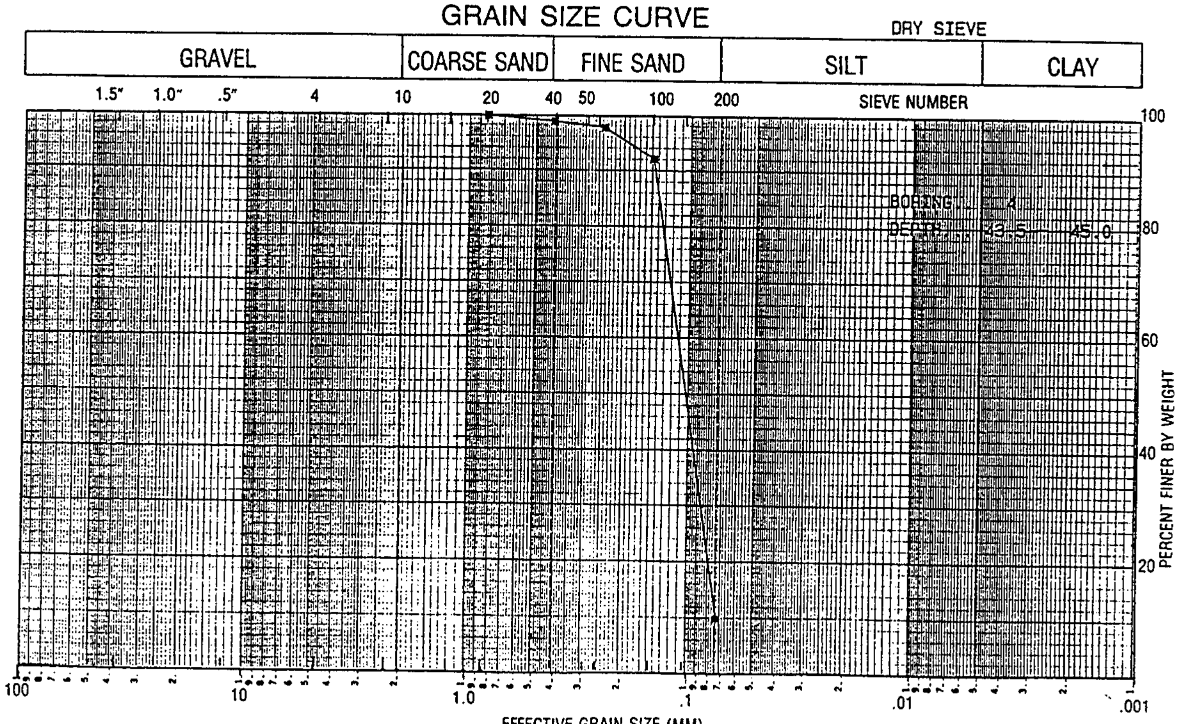
GRAIN SIZE CURVE



FILE 96- 56
FIGURE . . 27

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers

GRAIN SIZE CURVE

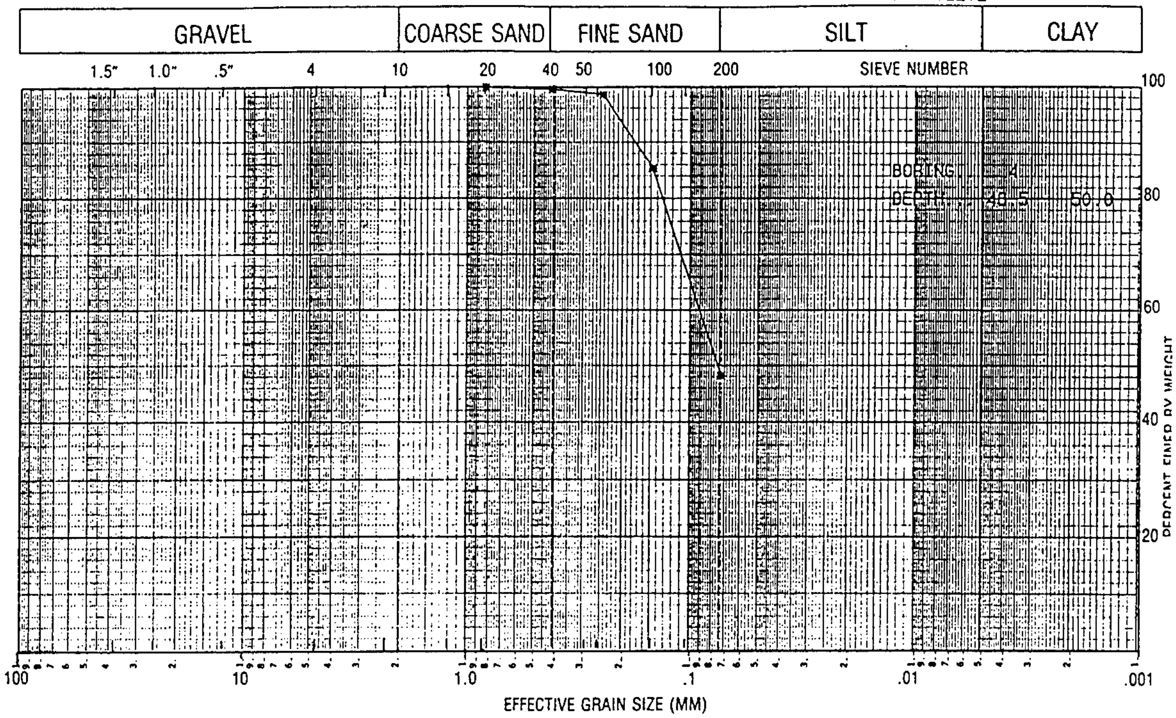


FILE 96- 56
FIGURE . . 28

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers

GRAIN SIZE CURVE

DRY SIEVE



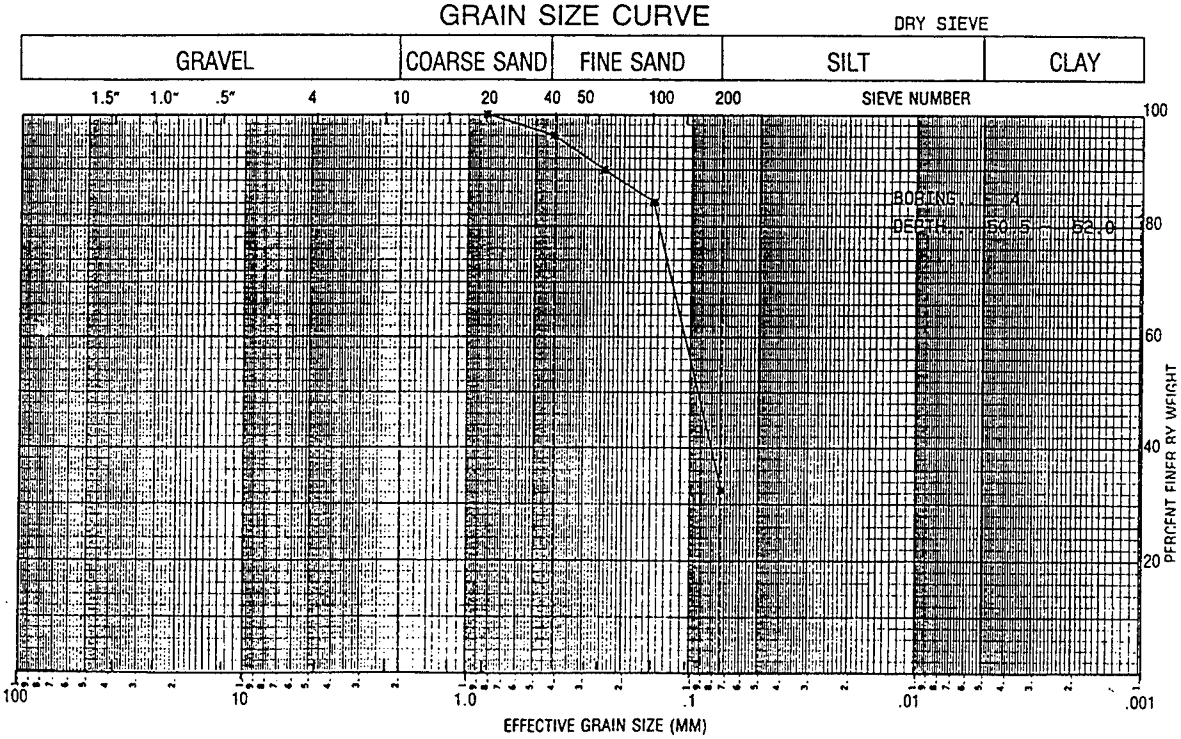
FILE 96- 56

FIGURE . . 29

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers

GRAIN SIZE CURVE

DRY SIEVE

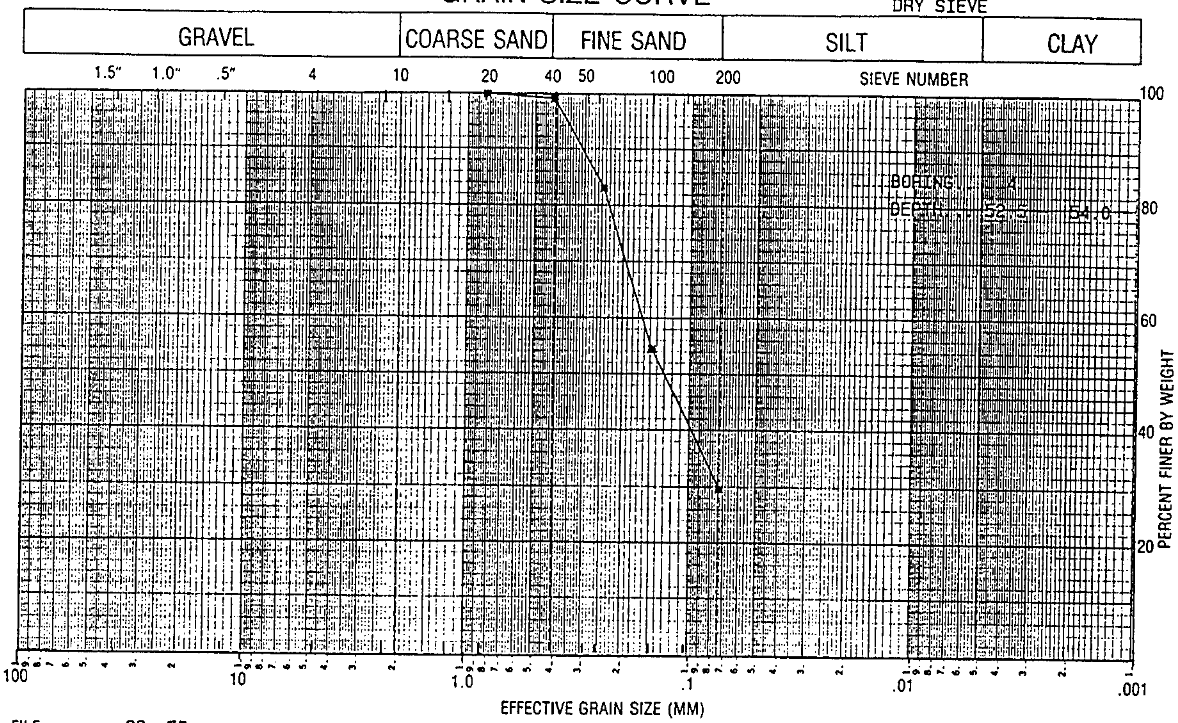


FILE 96- 56

FIGURE . . 30

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers

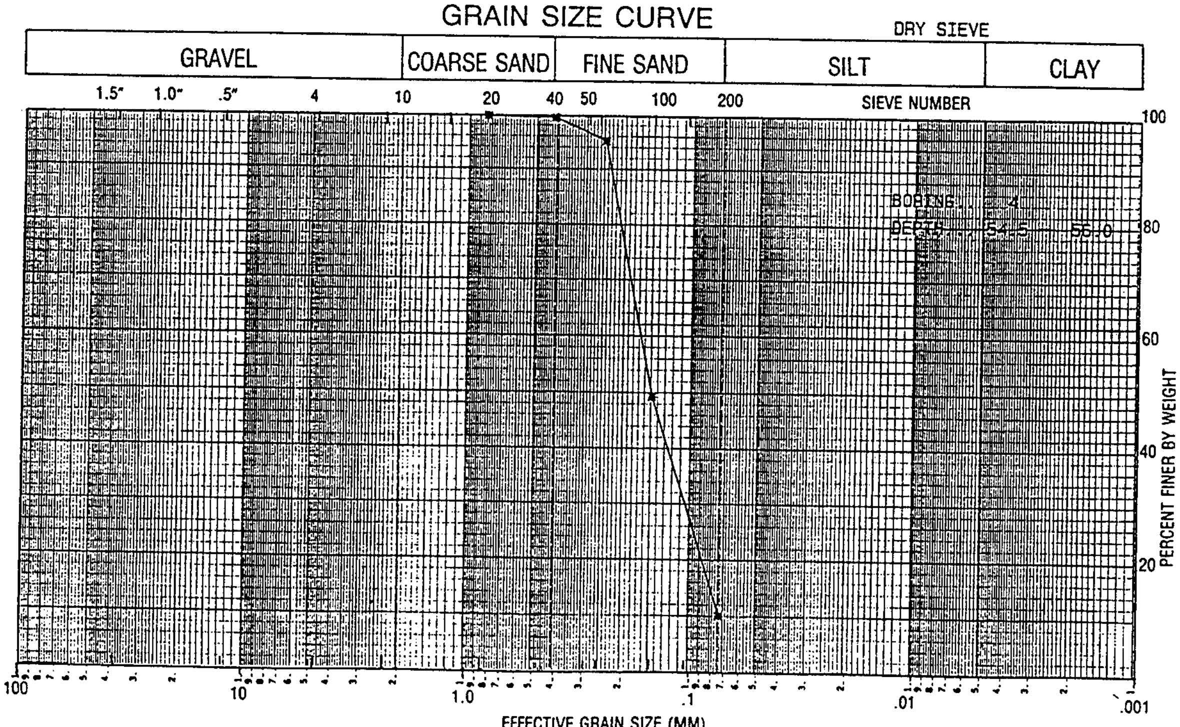
GRAIN SIZE CURVE



FILE 96- 56
FIGURE . . 31

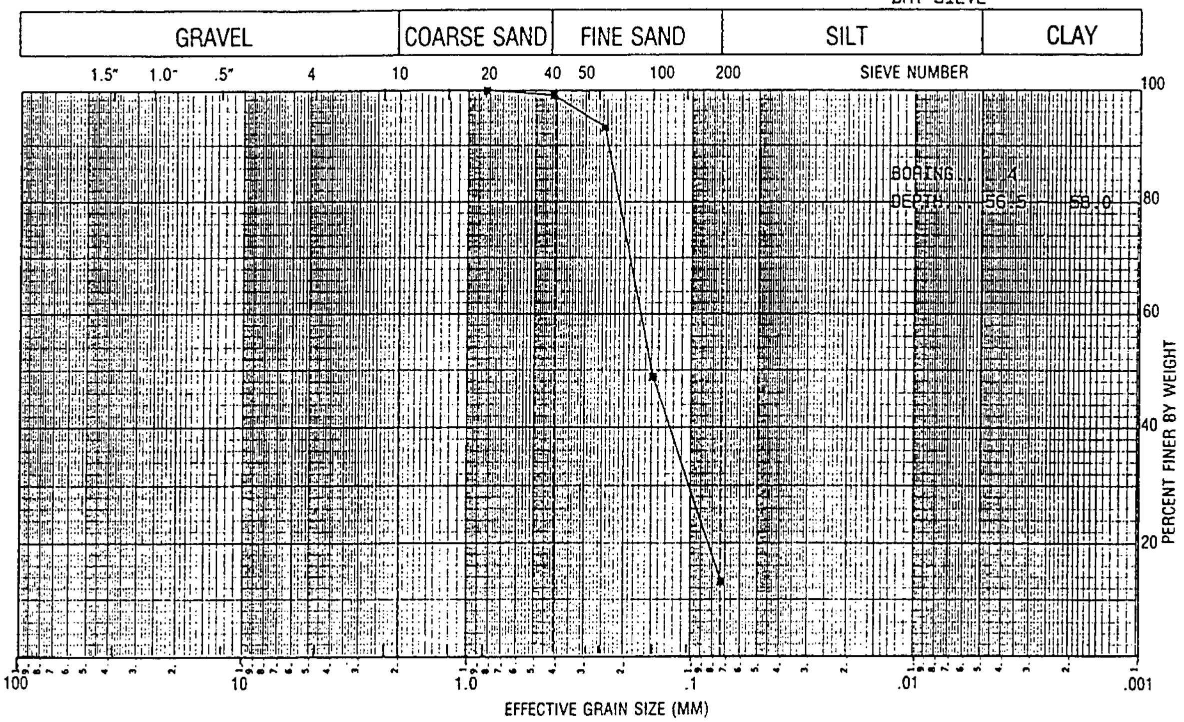
Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers

GRAIN SIZE CURVE



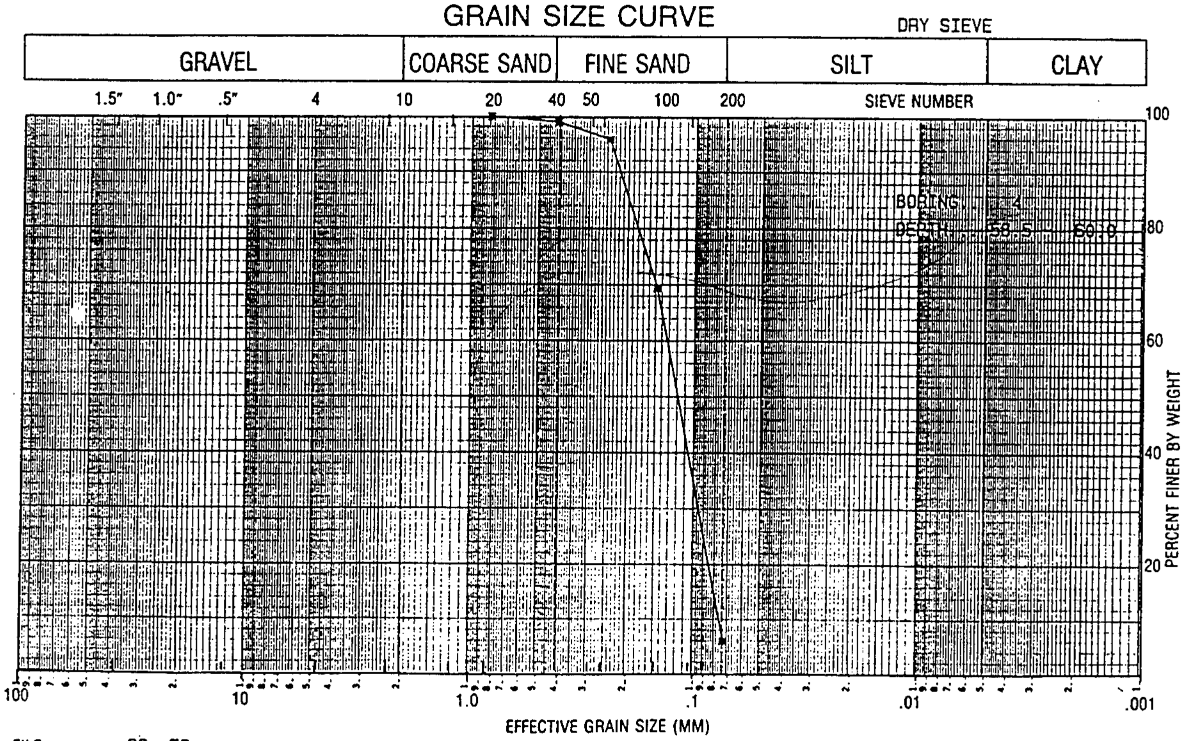
FILE 96- 56
FIGURE . . 32

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers



FILE 96- 56
FIGURE . . 33

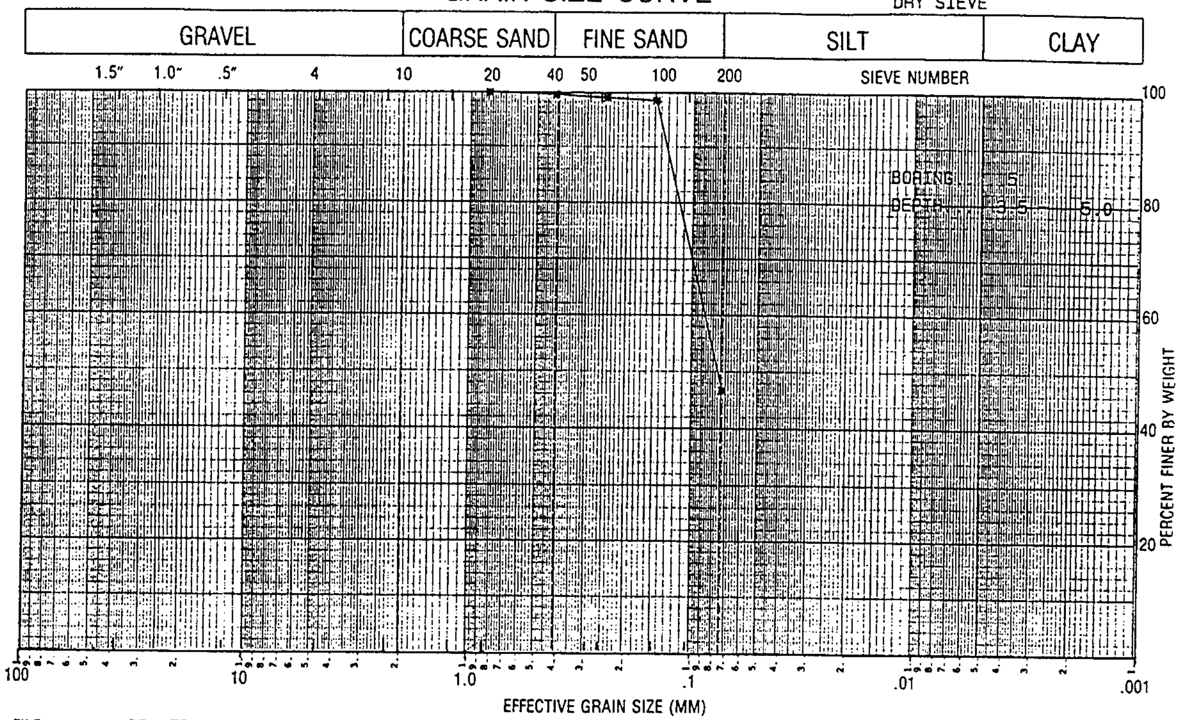
Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers



FILE 96- 56
FIGURE . . 34

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers

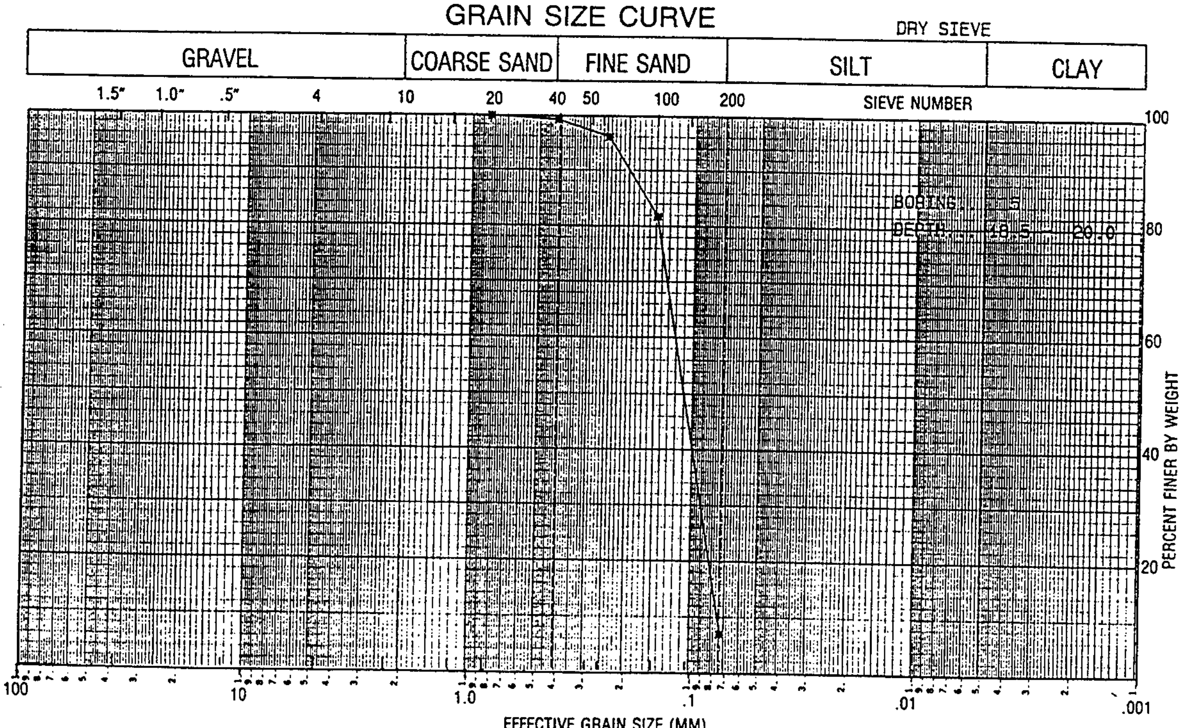
GRAIN SIZE CURVE



FILE 96- 56
FIGURE . . 35

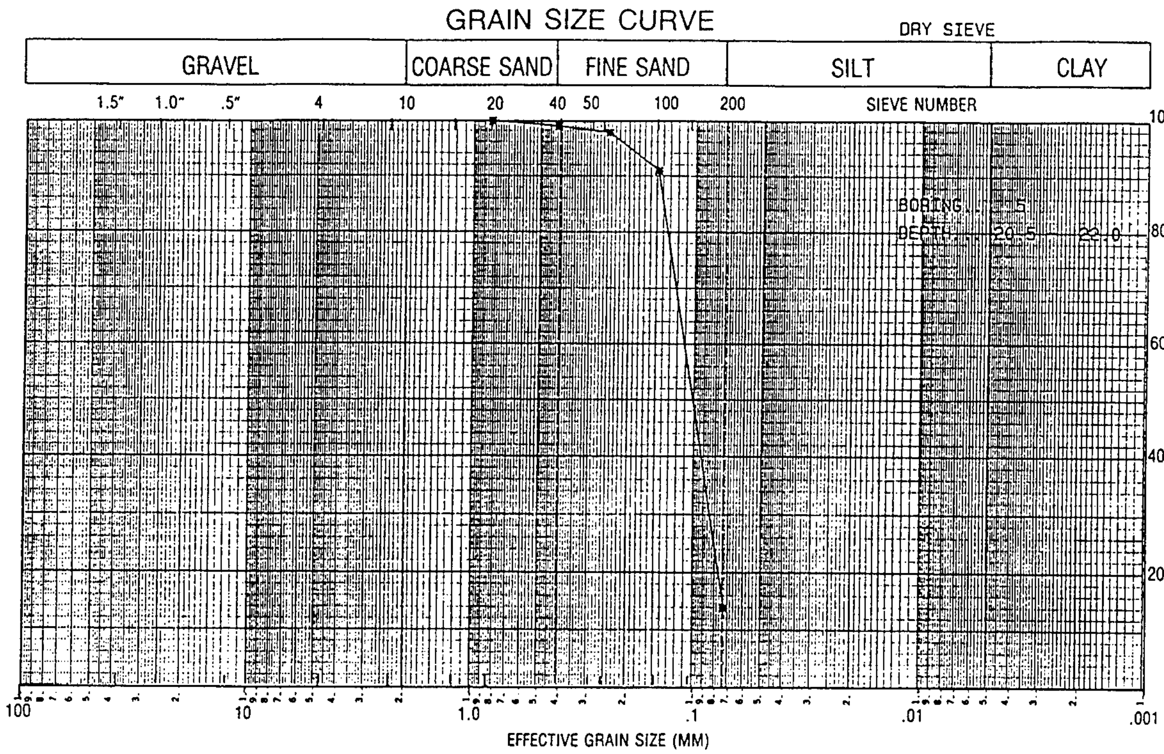
Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers

GRAIN SIZE CURVE



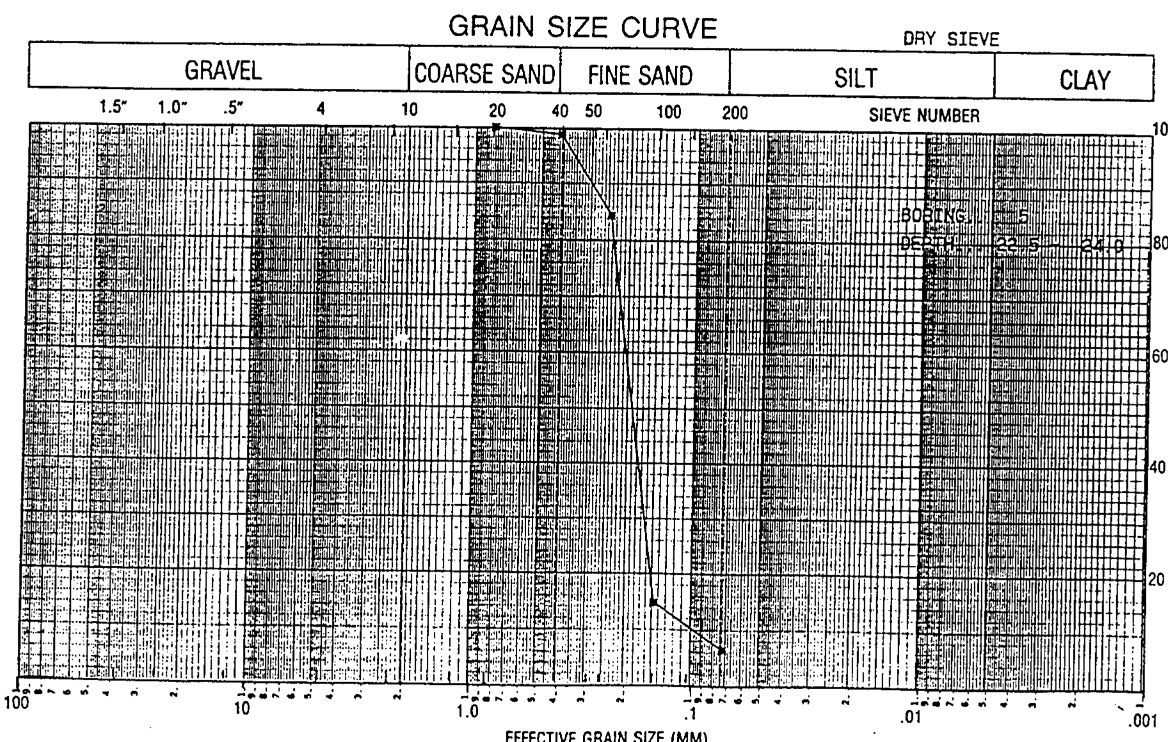
FILE 96- 56
FIGURE . . 36

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers



FILE 96- 56
FIGURE . . 37

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers

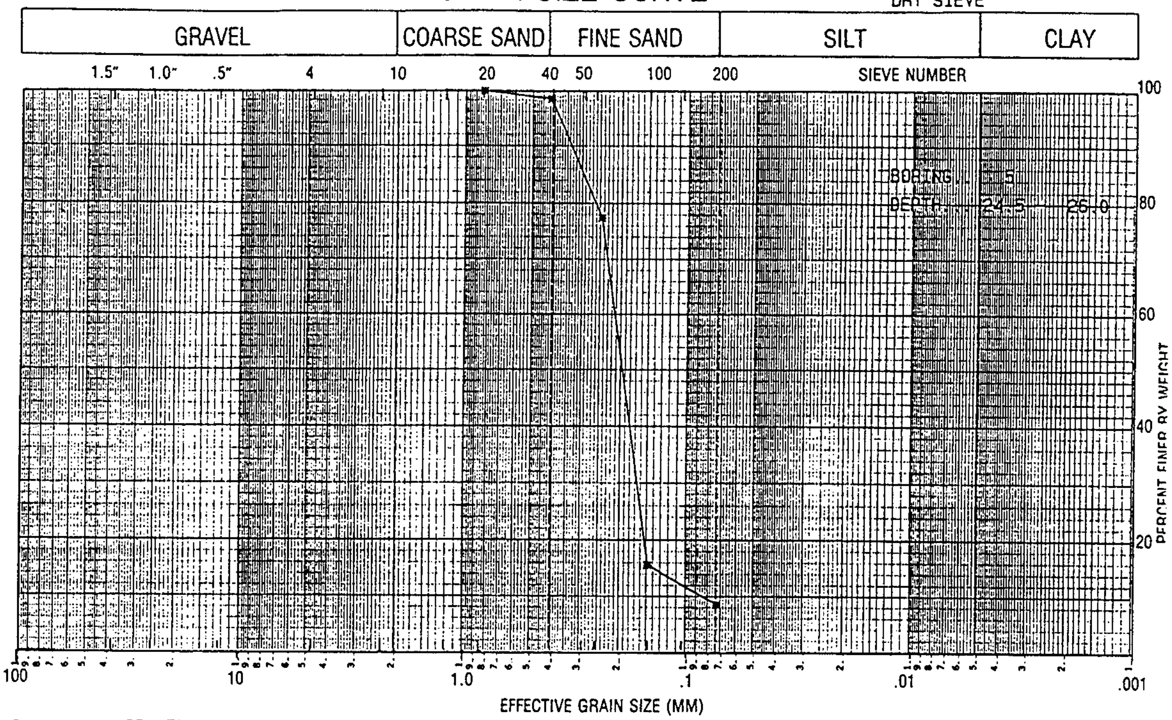


FILE 96- 56
FIGURE . . 38

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers

GRAIN SIZE CURVE

DRY SIEVE



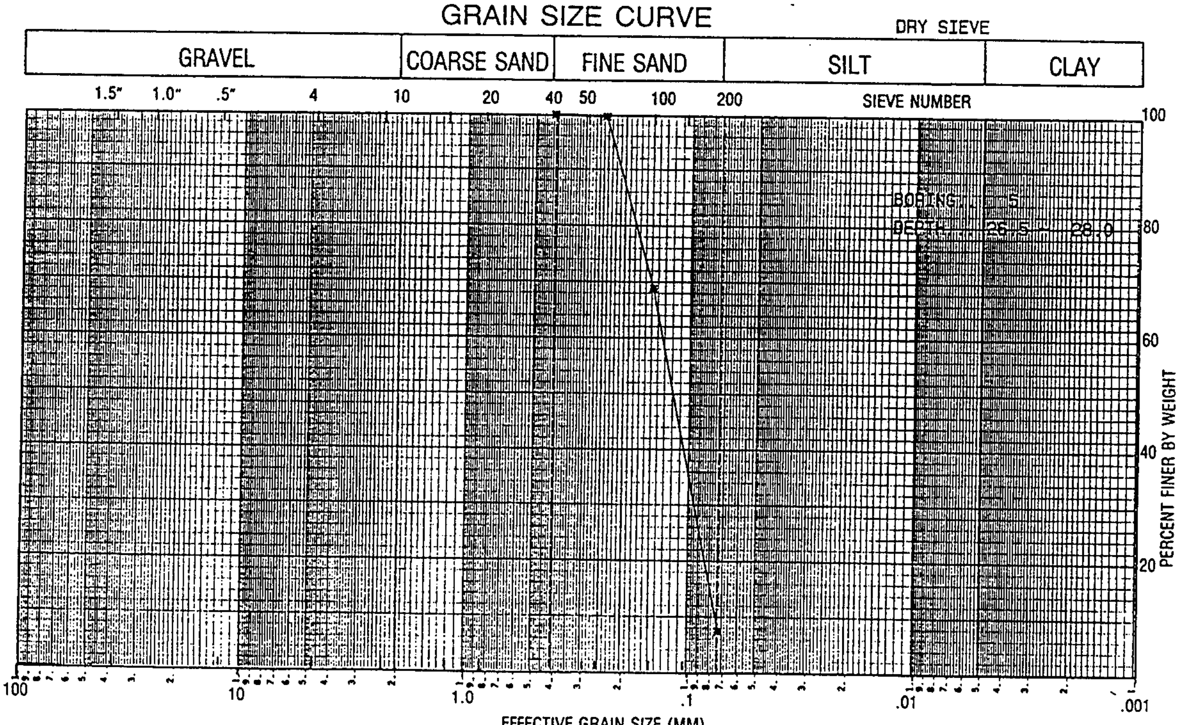
FILE 96- 56

FIGURE . . 39

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers

GRAIN SIZE CURVE

DRY SIEVE



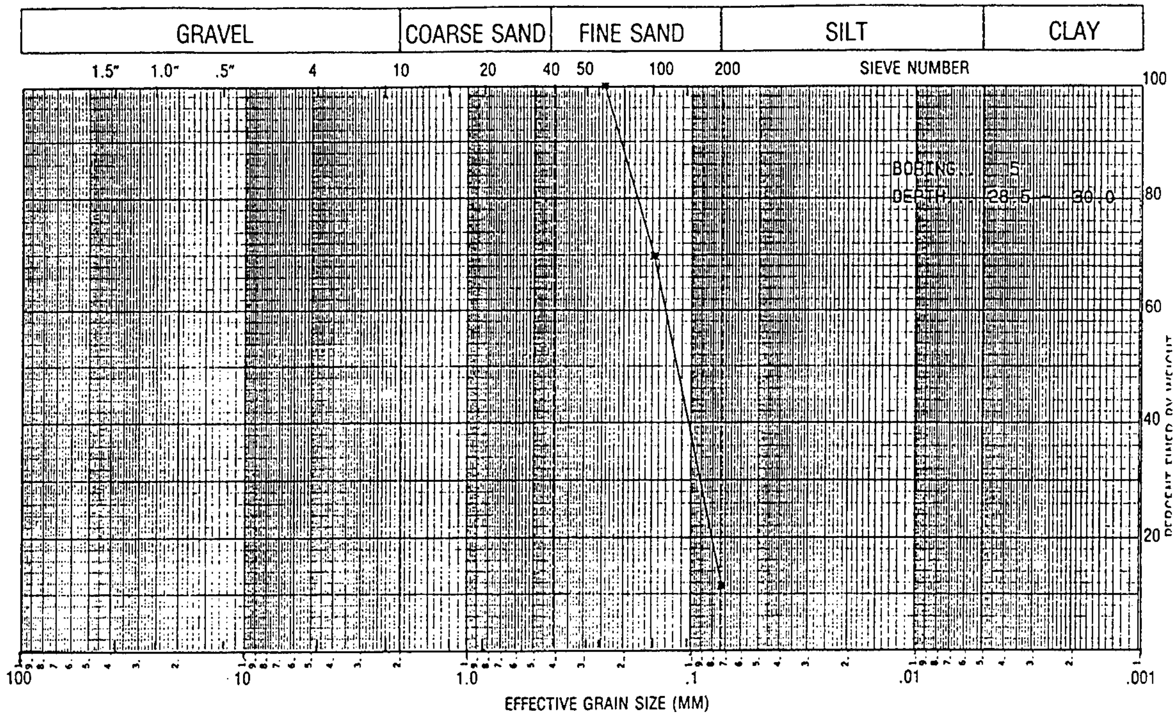
FILE 96- 56

FIGURE . . 40

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers

GRAIN SIZE CURVE

DRY SIEVE

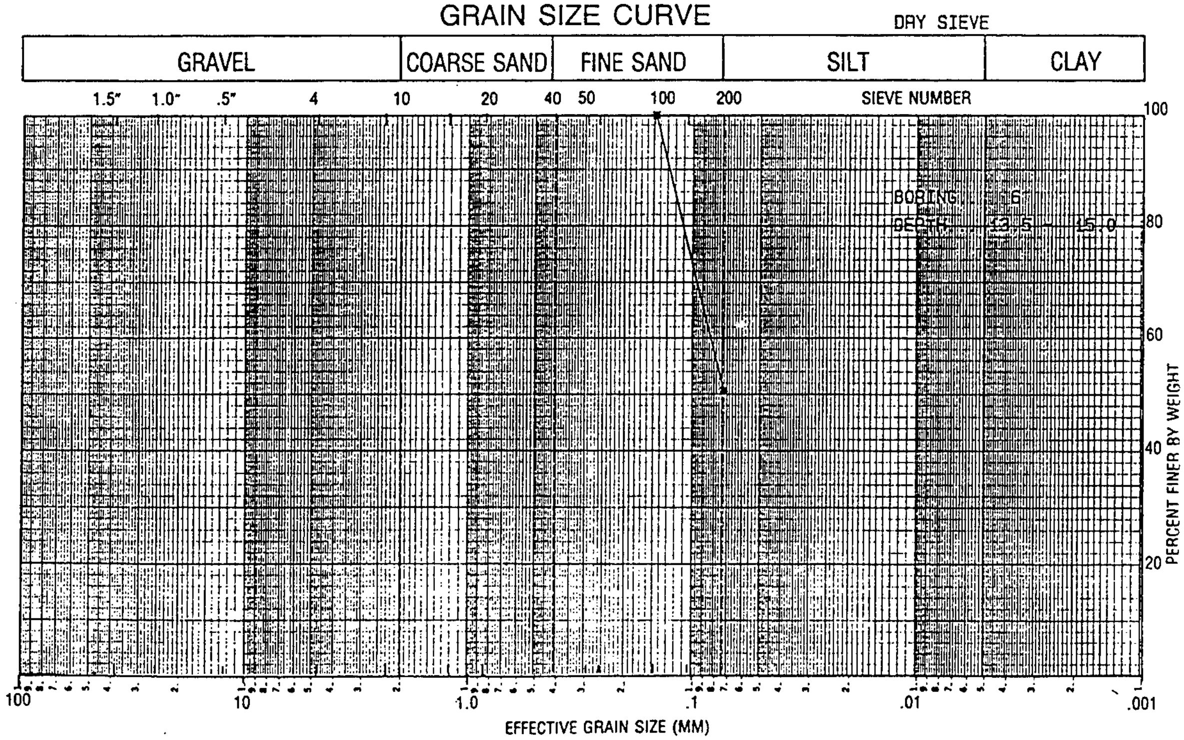


FILE 96- 56
FIGURE . . 41

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers

GRAIN SIZE CURVE

DRY SIEVE

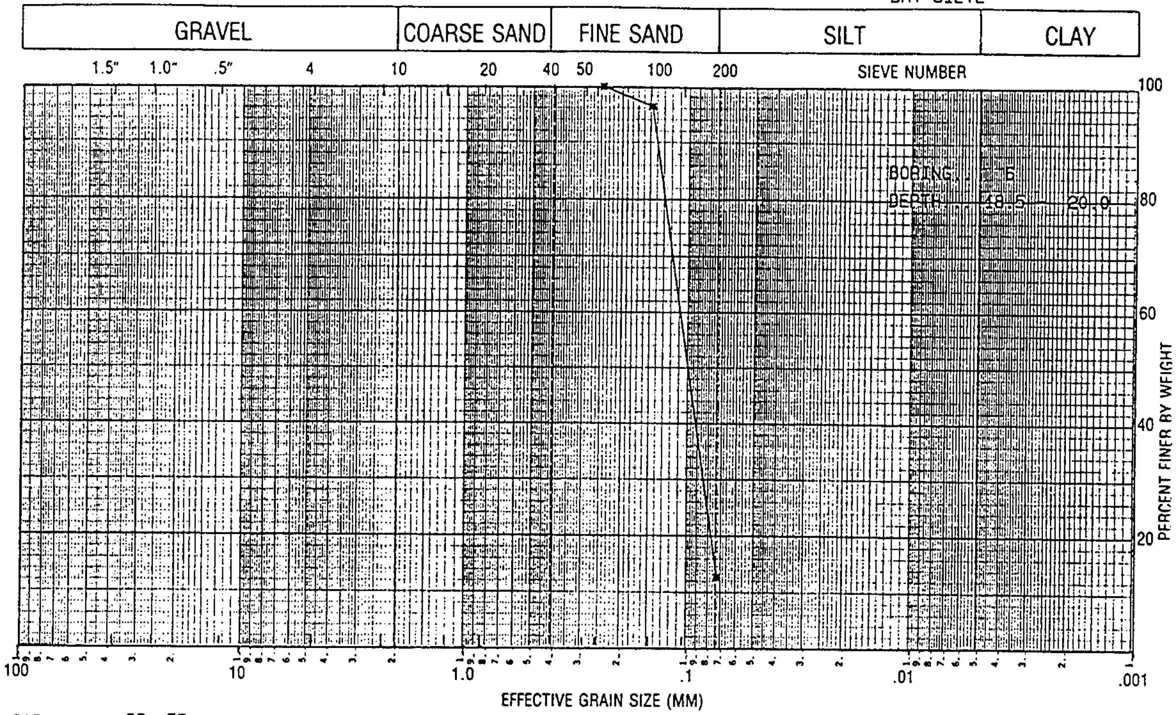


FILE 96- 56
FIGURE . . 42

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers

GRAIN SIZE CURVE

DRY SIEVE

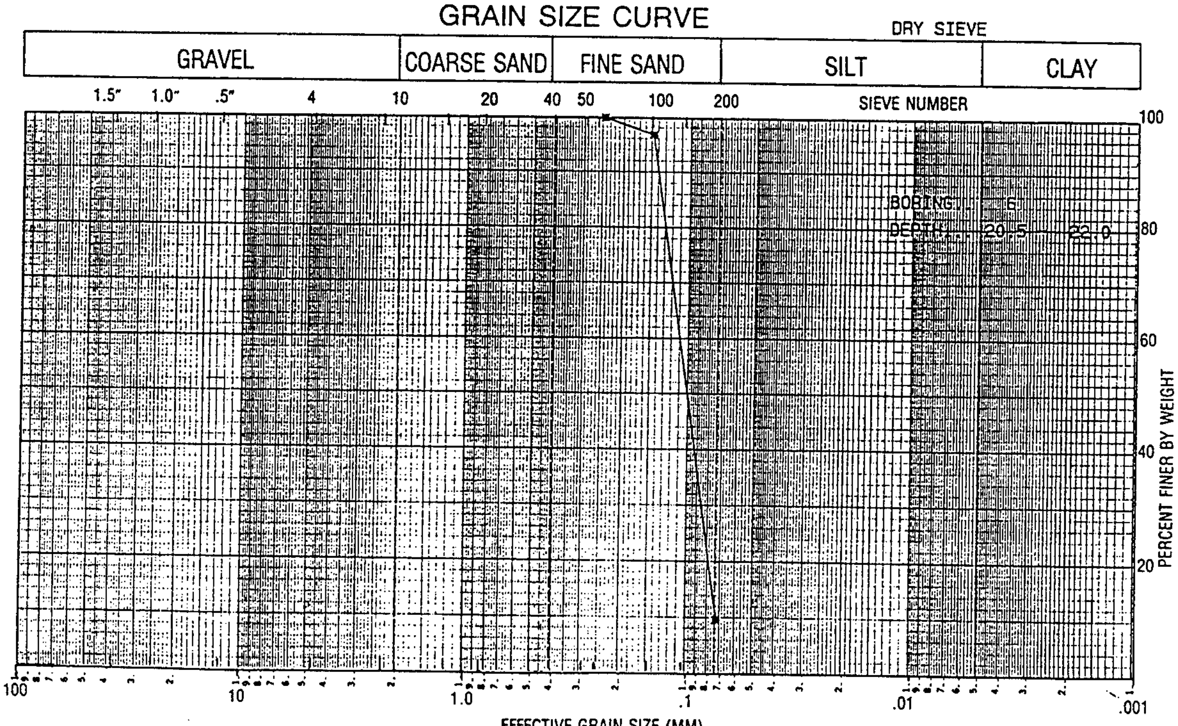


FILE 96- 56
FIGURE . . 43

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers

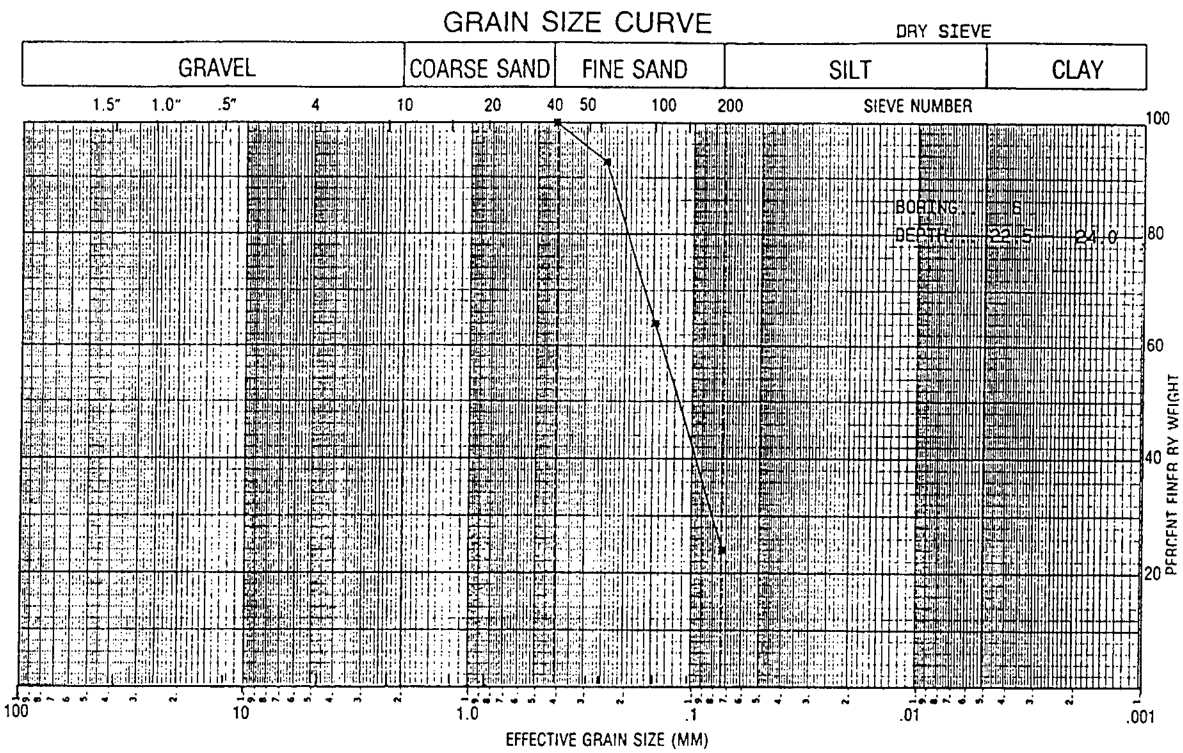
GRAIN SIZE CURVE

DRY SIEVE



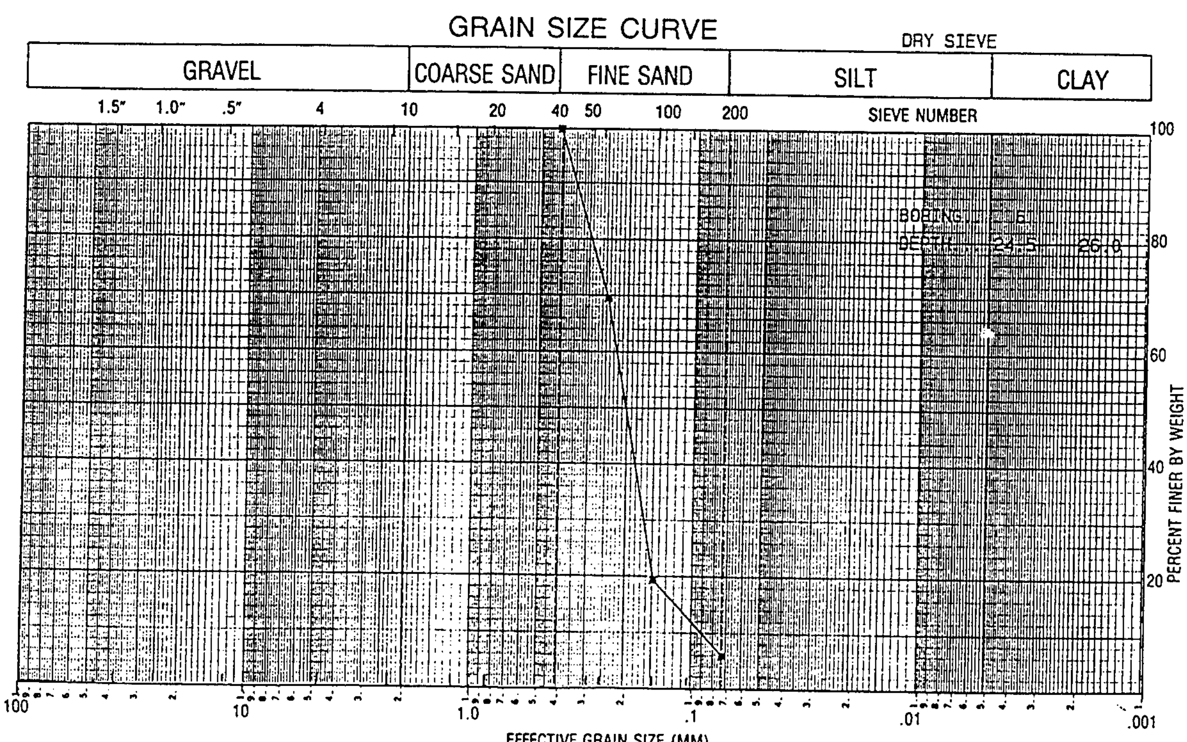
FILE 96- 56
FIGURE . . 44

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers



FILE 96- 56
FIGURE . . 45

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers

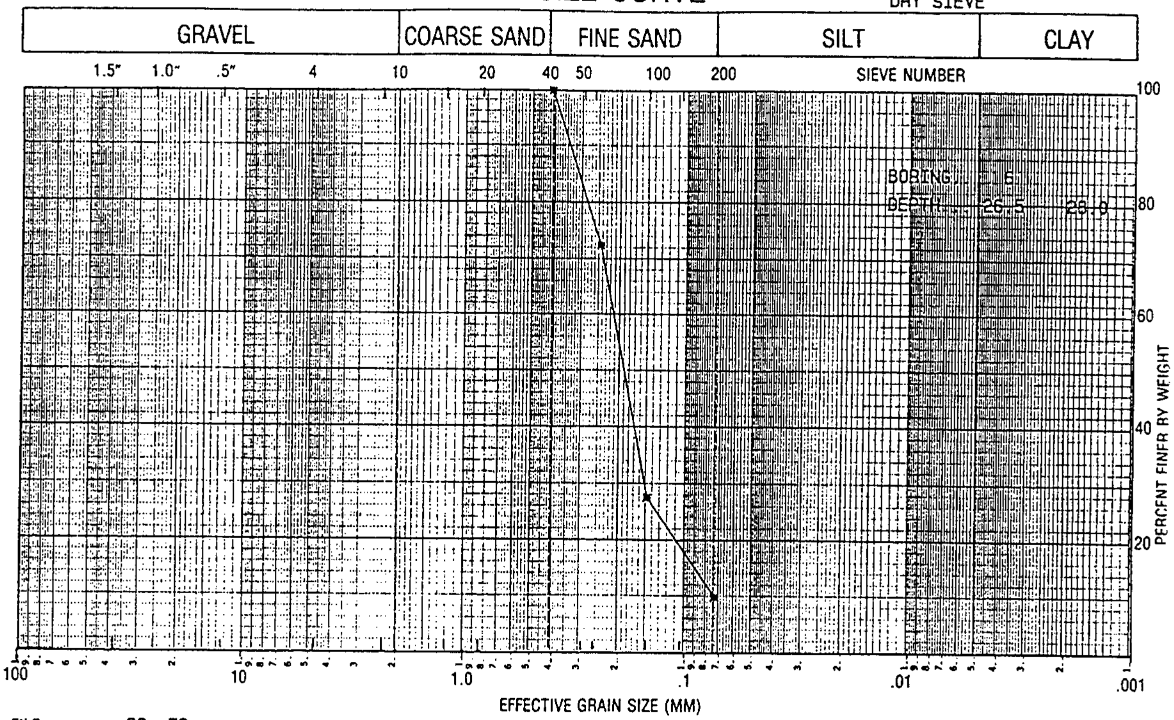


FILE 96- 56
FIGURE . . 46

Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers

GRAIN SIZE CURVE

DRY SIEVE



FILE 96- 56
FIGURE . . 47

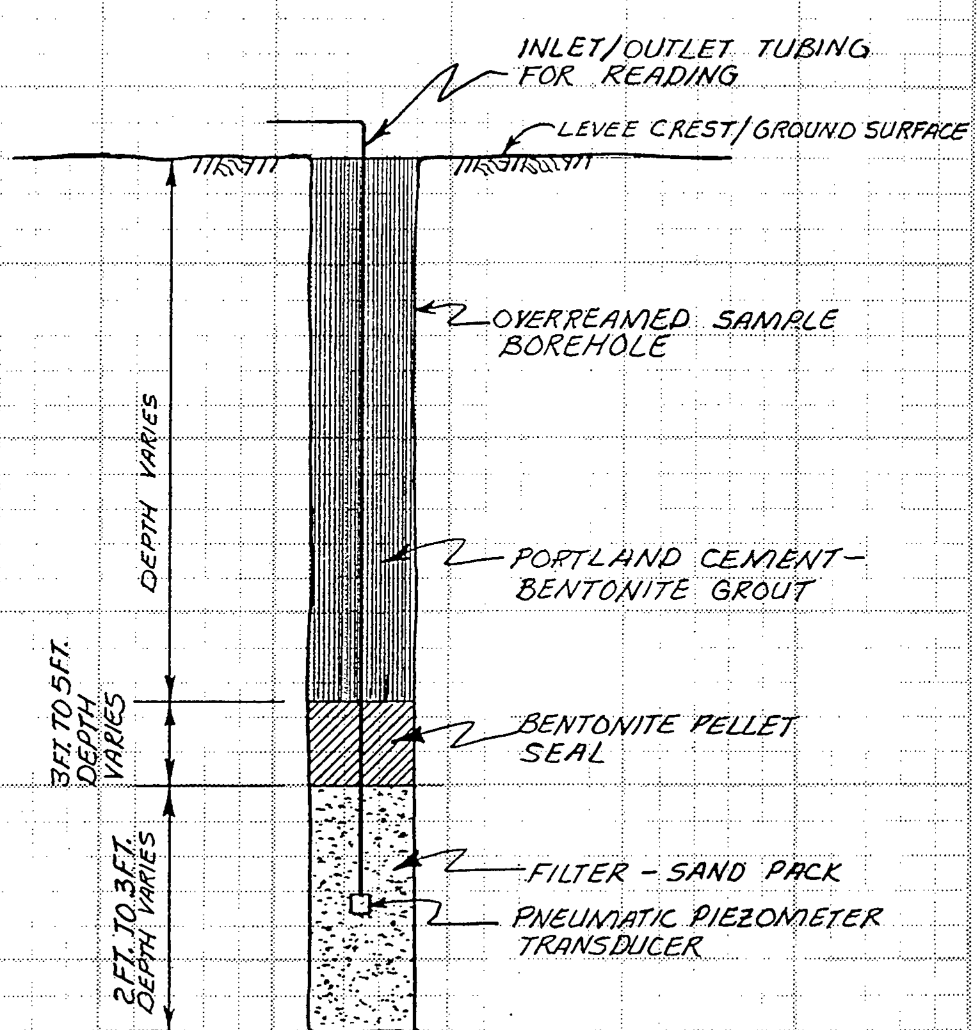
Louis J. Capozzoli and Associates, Inc.
Geotechnical Engineers

Subject PIEZOMETER SCHEMATIC

Made by BJB Date 30 MAY 96

File No. 96-56 Checked by DPS Date 30 May 96

LOUIS J. CAPOZZOLI & ASSOCIATES, INC. Geotechnical Engineers



PNEUMATIC PIEZOMETER INSTALLATION

LOG OF BORING

Project: Embankment Subdrilling Study
Issaquena County, Mississippi
For: US Army Corps of Engineers
Vicksburg, Mississippi

Boring: 1
File: 96-56
Date: 14 May 1996
Technician: ETG

Legend: ■ Undisturbed Sample; ☒ Standard Penetration Test; ◫ Classification Sample; (SLS) Slickensided

Boring Depth: 30 Feet

Depth Feet	Samples	Description	Penetration Resistance
0	Undisturbed Sample	Stiff gray clay with silty sand streaks, organic matter, and ferrous nodules	Piezometer #46607
5	Standard Penetration Test	Loose tan clayey sand Penetration Resistance	4 blows per foot (2/2/2)
10	Standard Penetration Test	Loose tan silty sand Penetration Resistance	5 blows per foot (4/2/3)
15	Undisturbed Sample	Firm gray sand with clay traces	
20	Undisturbed Sample	Firm gray sand with 1 1/2 inch sandy clay layer	
	Standard Penetration Test	Firm tan fine sand with silt traces Penetration Resistance	20 blows per foot (10/10/10)
	Standard Penetration Test	Firm tan silty fine sand Penetration Resistance	23 blows per foot (10/11/12)
25	Standard Penetration Test	Firm gray fine sand Penetration Resistance	25 blows per foot (12/12/13)
	Standard Penetration Test	Firm gray fine sand Penetration Resistance	22 blows per foot (10/11/11)
30	Standard Penetration Test	Loose gray sandy silt with 1/2 inch and 1 inch clay layer and fine sand traces Penetration Resistance	9 blows per foot (7/4/5)

After washing hole, sand slumped 5 feet in hole, set piezometer at 25 feet.

LOUIS J. CAPOZZOLI & ASSOCIATES, INC. Geotechnical Engineers

LOG OF BORING

Project:	Embankment Subdrilling Study Issaquena County, Mississippi
For:	US Army Corps of Engineers Vicksburg, Mississippi
Boring:	2
File:	'96-56
Date:	15 May 1996
Technician:	ETG

Legend: Undisturbed Sample; Standard Penetration Test; Classification Sample; (SLS) Slickensided

Boring Depth: 30 Feet

Depth Feet	Samples	Description	
0–1	Undisturbed Sample	Medium tan and gray sandy clay with sand streaks and ferrous nodules	Piezometer #46606
4–5	Standard Penetration Test	Loose tan silty sand with clay traces Penetration Resistance	5 blows per foot (2/2/3)
8–10	Undisturbed Sample	Firm gray sand with clay traces	
13–15	Undisturbed Sample	Firm gray sand with clay traces	
19–20	Standard Penetration Test	Firm tan fine sand with clay traces Penetration Resistance	14 blows per foot (5/6/8)
21–22	Standard Penetration Test	Firm tan fine sand Penetration Resistance	15 blows per foot (6/6/9)
23–24	Standard Penetration Test	Firm gray fine sand with silt traces Penetration Resistance	21 blows per foot (7/8/13)
25–26	Standard Penetration Test	Firm gray fine sand with silt traces Penetration Resistance	19 blows per foot (9/9/10)
27–28	Standard Penetration Test	Firm gray fine sand with silt traces Penetration Resistance	16 blows per foot (8/7/9)
28–30	Undisturbed Sample	Medium gray clay with sand streaks	

After washing hole, sand slumped 5 feet in hole, set piezometer at 28 feet.

LOUIS J. CAPOZZOLI & ASSOCIATES, INC. Geotechnical Engineers

LOG OF BORING

Project:	Embankment Subdrilling Study Issaquena County, Mississippi	Boring:	3
		File:	96-56
For:	US Army Corps of Engineers Vicksburg, Mississippi	Date:	15 May 1996
		Technician:	LRM

■ Undisturbed Sample
☒ Standard Penetration Test
◫ Classification Sample
(SLS) Slickensided

Boring Depth: 60 Feet

Depth Feet	Samples	Description	
0	■	Very stiff tan silty clay with roots	Piezometer #46608
	■	Firm tan and gray sand with clay traces	
5			
	■	Stiff gray clay with sand streaks and ferrous nodules	
10			
	■	Soft gray silty clay	
15			
	■	Firm gray clayey silt with 3 inches of loose tan sand at bottom	
20	☒	Loose gray clayey sand Penetration Resistance	9 blows per foot (6/5/4)
		No sample recovered	
25	☒	Firm tan clayey sand Penetration Resistance	11 blows per foot (5/5/6)
30	☒	Firm tan clayey sand Penetration Resistance	18 blows per foot (5/8/10)
35	☒	Firm tan clayey sand Penetration Resistance	20 blows per foot (9/8/12)
40	☒	Dense brown sand Penetration Resistance	35 blows per foot (9/14/21)
45	☒	Dense brown sand with silt traces Penetration Resistance	33 blows per foot (10/13/20)
50	☒	Firm gray clayey sand with silt traces Penetration Resistance	29 blows per foot (9/14/15)

LOUIS J. CAPOZZOLI & ASSOCIATES, INC. Geotechnical Engineers

LOG OF BORING

Project: Embankment Subdrilling Study	Boring: 3
Issaquena County, Mississippi	File: '96-56
For: US Army Corps of Engineers	Date: 15 May 1996
Vicksburg, Mississippi	Technician: LRM

Undisturbed Sample
Standard Penetration Test
Classification Sample
(SLS) Slickensided

Boring Depth: 60 Feet

Depth Feet	Samples	Description	
50		Dense gray clayey sand	
		Penetration Resistance	32 blows per foot (11/14/18)
		Dense gray fine sand with clay traces	
		Penetration Resistance	31 blows per foot (12/14/17)
55		Dense gray fine sand with silt and clay traces	
		Penetration Resistance	30 blows per foot (11/14/16)
		Dense gray fine sand	
		Penetration Resistance	34 blows per foot 12/16/18)
		Fine gray fine sand	
60		Penetration Resistance	29 blows per foot (10/14/15)

After washing hole, sand slumped 5 feet in hole, set piezometer at 58 feet.

LOUIS J. CAPOZZOLI & ASSOCIATES, INC. Geotechnical Engineers

LOG OF BORING

Project: Embankment Subdrilling Study
Issaquena County, Mississippi
For: US Army Corps of Engineers
Vicksburg, Mississippi

Boring: 4
File: 96-56
Date: 15 May 1996
Technician: LRM

■ Undisturbed Sample
☒ Standard Penetration Test
◫ Classification Sample
(SLS) Slickensided

Boring Depth: 60 Feet

Piezometer #46609

Depth Feet	Samples	Description	
0	■	Stiff tan silty clay	
	■	Firm tan clayey sand	
5			
	■	Loose brown slightly clayey sand	
10			
	■	Loose brown sand	
15			
	■	Firm gray sand with 1/4 inch clay layer	
20			
	■	Stiff gray clay	
25			
	■	Stiff gray clay	
30			
	☒	Firm gray fine sand Penetration Resistance	20 blows per foot (9/9/11)
35			
	☒	Firm gray fine sand with silt traces Penetration Resistance	26 blows per foot (8/12/14)
40			
	☒	Firm gray fine sand Penetration Resistance	28 blows per foot (9/14/14)
45			
	☒	Firm gray silty fine sand with clay traces Penetration Resistance	23 blows per foot (9/11/12)
50			

LOUIS J. CAPOZZOLI & ASSOCIATES, INC. Geotechnical Engineers

LOG OF BORING

Project: Embankment Subdrilling Study Issaquena County, Mississippi	Boring: 4
For: US Army Corps of Engineers Vicksburg, Mississippi	File: 96-56 Date: 16 May 1996 Technician: LRM

Legend: ■ Undisturbed Sample; ☒ Standard Penetration Test; ◫ Classification Sample; (SLS) Slickensided

Boring Depth: 60 Feet

Depth Feet	Samples	Description	
50	☒	Firm gray clayey fine sand Penetration Resistance	23 blows per foot (9/10/13)
	☒	Firm gray clayey fine sand Penetration Resistance	20 blows per foot (8/10/10)
55	☒	Fine gray fine sand Penetration Resistance	22 blows per foot (9/10/12)
	☒	Fine gray fine sand with silt traces Penetration Resistance	22 blows per foot (10/9/13)
60	☒	Firm gray fine sand Penetration Resistance	25 blows per foot (10/14/11)

After washing hole, sand slumped 5 feet in hole, set piezometer at 55 feet

LOUIS J. CAPOZZOLI & ASSOCIATES, INC. Geotechnical Engineers

LOG OF BORING

Project: Embankment Subdrilling Study
Issaquena County, Mississippi
For: US Army Corps of Engineers
Vicksburg, Mississippi

Boring: 5
File: 96-56
Date: 15 May 1996
Technician: ETG

Legend:
- ■ Undisturbed Sample
- ⊠ Standard Penetration Test
- ◫ Classification Sample
- (SLS) Slickensided

Boring Depth: 30 Feet

Depth Feet	Samples	Description	
0	■	Medium gray silty clay with roots	Piezometer #46604
5	⊠	Loose tan silty fine sand Penetration Resistance	7 blows per foot (3/3/4)
10	■	Firm tan sand with clay streaks	
15	■	Firm tan sand	
20	⊠	Firm tan fine sand with silt traces Penetration Resistance	22 blows per foot (8/10/12)
	⊠	Firm tan fine sand with silt traces Penetration Resistance	20 blows per foot (9/10/10)
	⊠	Firm tan fine sand Penetration Resistance	25 blows per foot (8/13/12)
25	⊠	Firm tan fine sand Penetration Resistance	23 blows per foot (9/10/13)
	⊠	Dense gray fine sand Penetration Resistance	38 blows per foot (14/18/20)
30	⊠	Dense gray fine sand with silt traces Penetration Resistance	37 blows per foot (16/19/18)

After washing hole, sand slumped 5 feet in hole, set piezometer at 30 feet.

LOUIS J. CAPOZZOLI & ASSOCIATES, INC. Geotechnical Engineers

LOG OF BORING

Project: Embankment Subdrilling Study Issaquena County, Mississippi	Boring: 6
	File: 96-56
For: US Army Corps of Engineers	Date: 15 May 1996
Vicksburg, Mississippi	Technician: ETG

Legend: ■ Undisturbed Sample; ☒ Standard Penetration Test; ◫ Classification Sample; (SLS) Slickensided

Boring Depth: 30 Feet

Depth Feet	Samples	Description	
0	Undisturbed	Medium tan silty clay with roots	Piezometer #46605
	Undisturbed	Stiff tan and gray clay with silt pockets	
5			
	Undisturbed	Loose tan fine sand	
10			
15	SPT	Loose gray sandy silt Penetration Resistance	9 blows per foot (4/4/5)
20	SPT	Firm tan fine sand with silt traces Penetration Resistance	12 blows per foot (5/5/7)
	SPT	Firm tan fine sand with silt traces Penetration Resistance	14 blows per foot (5/6/8)
	SPT	Firm gray silty fine sand Penetration Resistance	13 blows per foot (4/7/6)
25	SPT	Firm gray fine sand Penetration Resistance	19 blows per foot (6/8/11)
	SPT	Firm gray fine sand with silt traces Penetration Resistance	22 blows per foot (9/10/12)
	Undisturbed	Medium gray clay with 1/4 inch sand layer and 1/8 inch sand layers	
30			

After washing hole, sand slumped 5 feet in hole, set piezometer at 28 feet.

LOUIS J. CAPOZZOLI & ASSOCIATES, INC. Geotechnical Engineers

REPORT DOCUMENTATION PAGE

Form Approved
OMB No. 0704-0188

1.AGENCY USE ONLY *(Leave blank)*	2.REPORT DATE	3.REPORT TYPE AND DATES COVERED
	April 1998	Final report

4.TITLE AND SUBTITLE
Installation of Pipelines Beneath Levees Using Horizontal Directional Drilling

5.FUNDING NUMBERS

6.AUTHOR(S)
Kimberlie Staheli, David Bennett,
Hugh W. O'Donnell, Timothy J. Hurley

7.PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES)
U.S. Army Engineer Waterways Experiment Station
3909 Halls Ferry Road, Vicksburg, MS 39180-6199

O'Donnell Associates, Inc.
P.O. Box 1208, Sugarland, TX 77487

8.PERFORMING ORGANIZATION REPORT NUMBER
CPAR-GL-98-1

9.SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES)
U.S. Army Corps of Engineers
Washington, DC 20314-1000

10.SPONSORING/MONITORING AGENCY REPORT NUMBER

11.SUPPLEMENTARY NOTES
Available from National Technical Information Service, 5285 Port Royal Road, Springfield, VA 22161.

12a.DISTRIBUTION/AVAILABILITY STATEMENT
Approved for public release; distribution is unlimited.

12b.DISTRIBUTION CODE

13.ABSTRACT ***(Maximum 200 words)***

Three horizontal directionally drilled bores were completed beneath a Mississippi River levee near Vicksburg, MS. Operating parameters were varied in a controlled fashion and ground conditions were carefully documented. Changes in pore pressures were monitored before, during, and after installation to help assess impacts on stability. Internal and external drilling fluid pressures were continuously monitored. An exhaustive autopsy was conducted by excavating down to the installed pipelines to determine the zone of influence around the pipelines created by the drilling fluids. dye tracers were added to the drilling fluids to allow visual determination of fluid migration. Important conclusions from this investigation were that regardless of the internal drilling fluid pressures measured at the nozzle, which ranged from 10.6 to 24.7 kg/cm2 (150 to 350 psi), the external drilling fluid pressures, measured in the annular space 0.305 m (1 ft) behind the nozzle remained within a narrow range of 3.3 to 3.7 kg/cm2 (47 to 52 psi), or 15 to 31 percent of the internal pressures. The increases in piezometric pressures were less than 1 percent of these external pressures, and less than 0.2 percent of the internal pressures. These small increases in pressures should be of no concern with regard to levee stability.

14.SUBJECT TERMS

Drilling pressures	HDD	Pilot bore
Guided drilling	Horizontal directional drilling	Pneumatic piezometers
Guidelines for HDD	Levee crossings	Reaming

15.NUMBER OF PAGES
194

16.PRICE CODE

17.SECURITY CLASSIFICATION OF REPORT	18.SECURITY CLASSIFICATION OF THIS PAGE	19.SECURITY CLASSIFICATION OF ABSTRACT	20.LIMITATION OF ABSTRACT
UNCLASSIFIED	UNCLASSIFIED		

NSN 7540-01-280-5500

Standard Form 298 (Rev. 2-89)
Prescribed by ANSI Std. Z39-18
298-102